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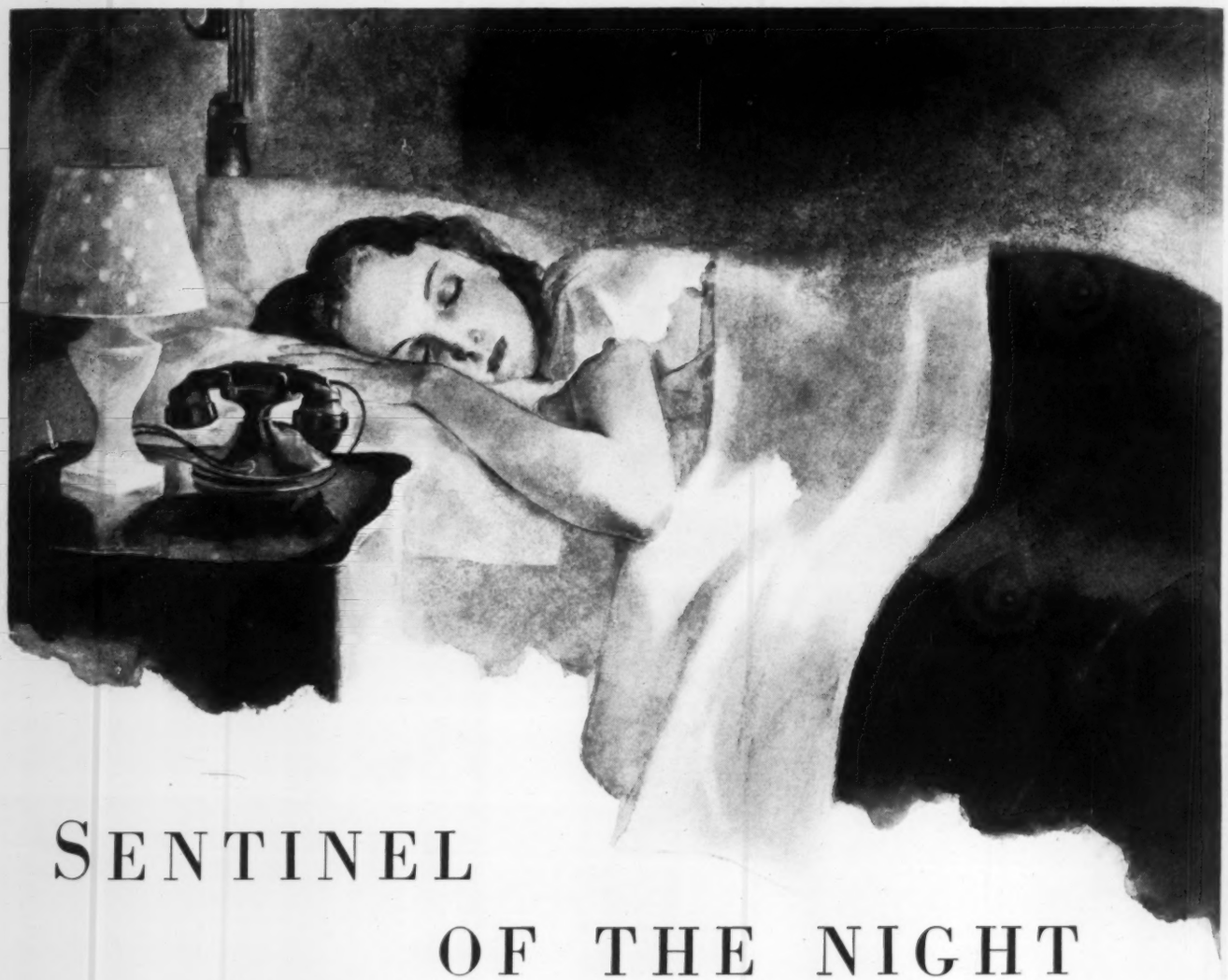

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Among Our Writers

FRANK A. KITTREDGE has had wide experience in highway engineering—with the State Highway Departments of Washington, Oregon, and California between 1907 and 1917; with the U.S. Bureau of Public Roads from 1917 to 1927; and as Chief Engineer of the National Park Service, since 1927.

L. I. HEWES, after receiving his Ph.D. degree at Yale in 1901, became Professor of Mathematics and Engineering at Rhode Island State College, and later instructor at Yale. Since 1911 he has been connected with the U.S. Bureau of Public Roads.

HERMAN STABLER has been engaged in the work of the U.S. Geological Survey and Reclamation Service connected with the problems of the public domain almost continuously since 1903. Since 1925 he has also been in charge of enforcement of the mineral leasing laws for the Survey.

E. B. DEBLER joined the U.S. Bureau of Reclamation in 1918 after varied engineering experience. Since 1922 he has directed most of the investigations of the Bureau's projects, particularly those connected with the determination of proper reservoir, spillway, and canal capacities, and extent and character of power output at proposed plants.

CONDE B. MCCULLOUGH has had broad professional experience in the field, in the office, and as a professor of civil engineering. Since 1919 he has been Bridge Engineer for the Oregon State Highway Department. He is the author of several books on bridges.

J. G. MASON, after practical and teaching experience in structural engineering, established a consulting practice in Lincoln, Neb. Since 1926 he has served as State Bridge Engineer for the Nebraska Department of Public Works.

A. L. OGLE, as Project Engineer for the Nebraska Department of Public Works, had charge of construction of the Plattsmouth Platte River Bridge, an early type using steel pile foundations. He has served for the last seven years as assistant bridge engineer in the bridge department.

J. W. BERETTA gained his early experience in the employ of the American Bridge Company, serving as Resident Engineer for the Delaware River Bridge, in Philadelphia. In 1927 he established the firm of J. W. Beretta Engineers, Inc., in San Antonio, Tex., having a general consulting practice.

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W. H. GARDINER, both as engineer and contractor, was for many years engaged on railroad construction in the Northwest. He became Construction Superintendent for Winston Brothers Company in 1921 and is now manager of the firm's Seattle branch and a member of the board of direction.

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D. C. HENNY, as supervising engineer for the U.S. Bureau of Reclamation, designed and built several earth and masonry dams in the Northwest. Since 1909 he has been connected as consultant with nearly all the dams built by the Bureau and is at present a member of the Board of Consulting Engineers on Hoover Dam.

VOLUME 2 NUMBER 9

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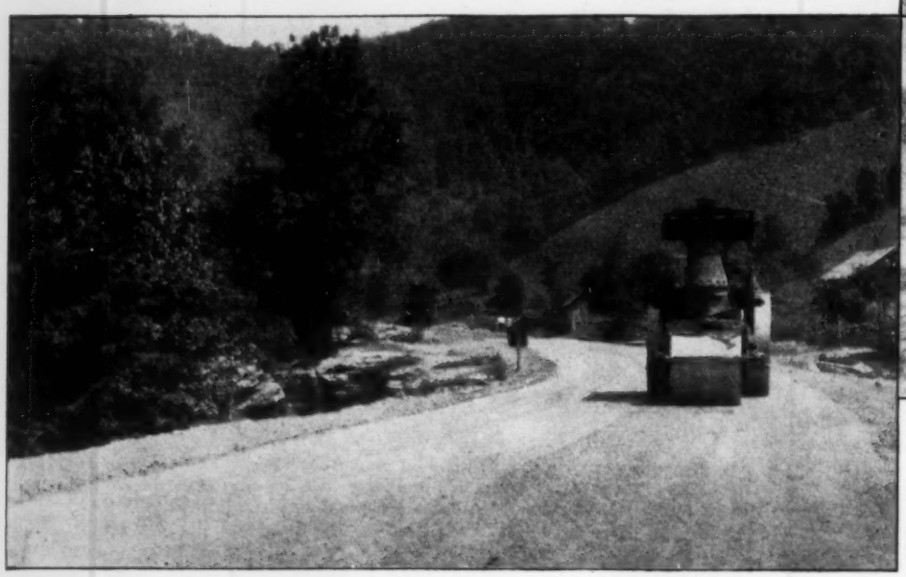

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VOLUME 2

NUMBER 9

The National Parks and Their Improvement

Special Methods Provide Maximum Accessibility While Leaving Nature Unspoiled

CERTAIN areas of unparalleled scenic beauty, geological significance, or historical value have been designated as National Parks or National Monuments, and set aside for preservation. But something more is needed in order that the public may enjoy these parks in comfort; many facilities must be provided to care for the millions of tourists who visit them each year.

Papers covering two phases of this work, presented at the Annual Convention of the Society in Yellowstone National Park, have been abstracted here. The article by Mr. Kittredge, prepared from the paper read by him on July 6, before the general technical meeting, gives a description of the parks, and discusses various features, many of them unique, of the engineering work of the National Park Service. The article by Dr. Hewes, whose complete paper was read on July 7, before the joint session of the Highway and Structural Divisions, treats of the part played by the U.S. Bureau of Public Roads in constructing highways for the National Park Service. Since 1924, when this phase of its work began, one-third of a carefully planned system of 1,800 miles of park highways has been completed. Dr. Hewes explains the principles on which these roads, and the necessary bridges and tunnels, have been designed and built.

Preserving a Valuable Heritage

By FRANK A. KITTREDGE
MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS
CHIEF ENGINEER, NATIONAL PARK SERVICE, U.S. DEPARTMENT OF THE INTERIOR,
SAN FRANCISCO, CALIF.

THE term "improvements" in connection with National Parks is considered by some to be a misnomer. There are those among outdoor enthusiasts who say that Nature's work cannot be improved, and that anything man can do is destructive and hence not an improvement.

The Act of Congress of August 25, 1916, creating the National Parks, states that these areas shall be set aside to "conserve the scenery and the natural and historic objects and the wild life therein; and, to provide for the enjoyment of the same in such manner and by such means as will leave them unimpaired for the enjoyment of future generations." The directors of the National Park Service, beginning with Stephen T. Mather in 1916, and continuing with Horace M. Albright since 1929, have established and followed policies that appeared, in so far as possible, to pursue the practical aim of making the wonderful scenery and geological features available to the greatest number of people, with the minimum change of topography. Secretary Ray Lyman Wilbur, with his broad practical knowledge of outdoor problems, and Director Horace M. Albright have been able to carry on policies which will aid in conserving for all time the unique park areas for the enjoyment of future generations.

Improvements in the National Parks are not of great magnitude, but they do have complicated technical features. The construction of a water supply system for the Old Faithful area, the satisfactory disposal of sewage for each of the public areas in all the parks, the proper lighting of such areas, and the provision of adequate roads, are all of great importance to the communities involved and to the park-visiting public.

Some of these projects are handled cooperatively with the assistance of the War Department; some, through the U.S. Public Health Service; and a large number through the Bureau of Public Roads, which takes care of all the major road construction. The great mass, in number of projects though not in terms of dollars spent, is handled by the engineering staff of the National Park Service. This staff is versatile in its make-up, and necessarily so, for its construction projects cover trails and minor roads, buildings, sewage plants, water supply, including the drilling of deep wells, hydro-electric and Diesel power plants, passenger elevators, drainage, special lighting, river erosion and flood control, and many other projects, some too strange to receive credence. Much of this work is done by day labor directed by the staff of the Park Service. The cost of the various undertakings runs from a few hundred dollars to many thousands.

Improvements in the National Parks represent an expenditure during the last seven years of $35,000,000, allocated among approximately 2,000 projects scattered from Hawaii to Maine, and from Mt. McKinley, Alaska, to the Mexican border, in each of which the project

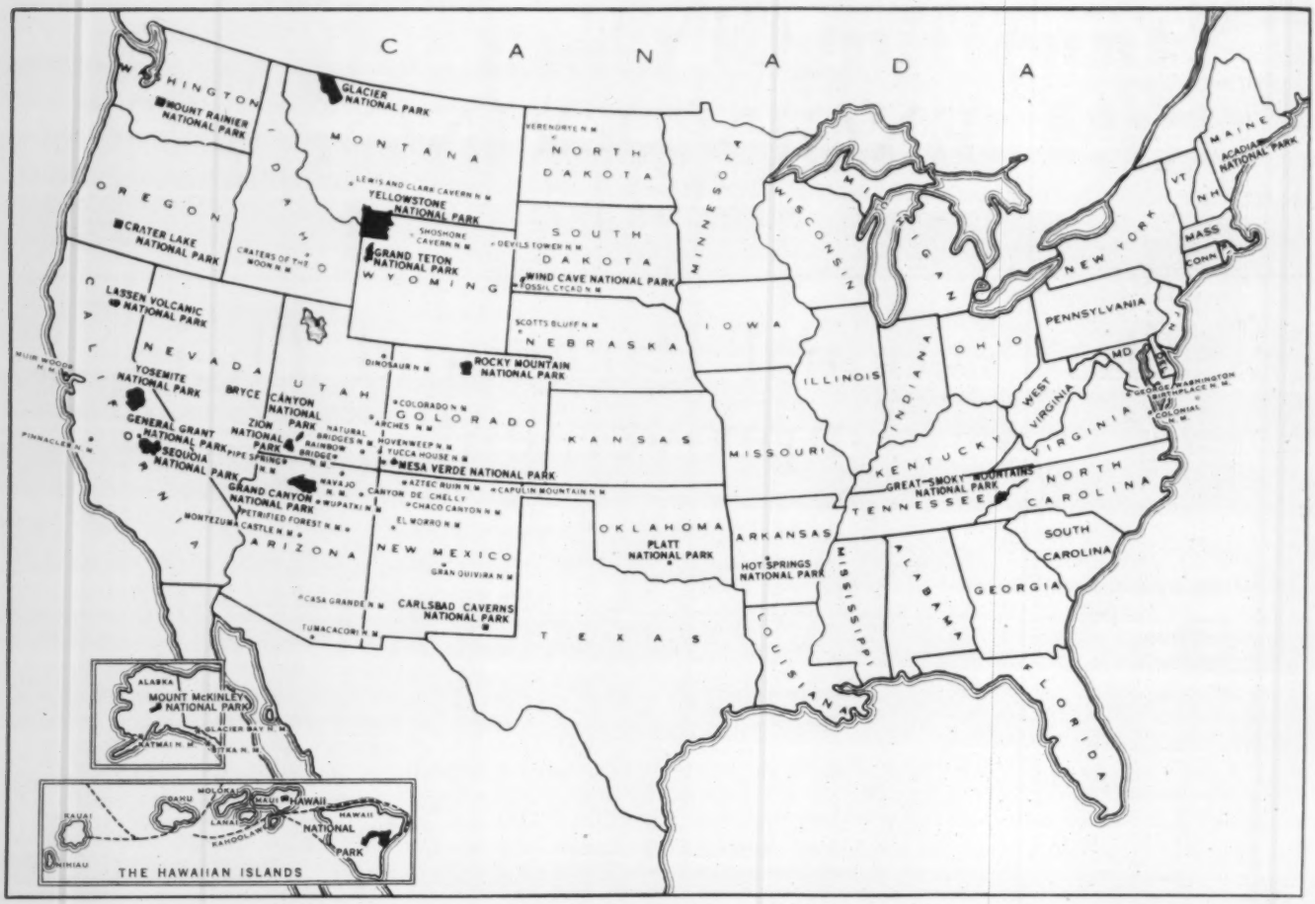


FIG. 1. LOCATION OF 22 NATIONAL PARKS AND 34 NATIONAL MONUMENTS

engineer must have and exercise initiative, judgment, and engineering skill, without close and detailed supervision.

The Park Service is operating at an educational and inspirational profit to the individuals who use the parks, and thus for the benefit and general welfare of the country at large. Referring to Fig. 1, it will be noted that most of the parks lie in the West.

The number and location of the National Parks in the various states are not determined by political preference. The choice of an area as a National Park is dependent entirely on its worthiness. The policy of establishing National Parks or Monuments only for unique localities, either scenic, historic, geologic, or archeologic, may be a new thought to some. It is only recently that the National Park system has been made to include historical ground. Washington's birthplace and the Battle of Yorktown area, on the Atlantic Coast, have now been included.

NATIONAL PARKS DESCRIBED

Beginning in the northwest corner of the map, Fig. 1, Mount Rainier Park was set aside because it contained the most outstanding example of a single glaciated mountain peak in the United States. It was only later that Mt. McKinley, in Alaska, nearly 6,000 ft. higher than Mt. Rainier, came to be generally known.

Glacier National Park was chosen primarily because it contains some of the noblest mountain scenery in America. In ruggedness and sheer grandeur it probably surpasses the Alps, although geologically it is markedly different. The geological structure, as shown by the Lewis overthrust, is vastly older and much more richly colored than any other rock in America excepting that in the Grand Canyon.

Yosemite National Park, besides comprising a part of the High Sierra mountain range, includes also the most spectacular example of a large glaciated valley. Many people think of Yosemite Park as comprising only this valley, which is, however, merely a crack seven miles long in a scenic masterpiece of 1,100 square miles.

The Grand Canyon is the world's most spectacular illustration of erosion—of the combined action of running water, rain, wind, and the various atmospheric agents that take the rocks and sculpture them into forms that give character to a landscape. The Colorado River flows through this gash in the plateau 8,000 ft. deep and 10 miles wide at the top.

Sequoia National Park contains the finest specimens of the Giant Sequoia tree. One can hardly comprehend the great diameter of these trees when viewing them because of the large size of all the trees in the vicinity. The General Sherman Tree is 37 ft. in diameter, and there are scores of others almost as large. Hundreds of them are over 10 ft. in diameter.

Carlsbad Caverns National Park is outstanding as an example of certain wonderful formations. No other known cavern contains the great variety of stalactites, stalagmites, helictites, and the various modifications of limestone formations exhibited there.

Yellowstone Park is the most widely celebrated of all. Its high plateau areas, its geysers, its canyons, its great variety of wild animals, and other unusual features, give it a broader field of interest than probably any other park.

The Southwestern Monuments area contains a great many of the prehistoric ruins in the United States. Overhanging one of the largest prehistoric villages, Pueblo Bonito, is a huge cliff, which apparently had cracked away from the main mass during the occupancy

of the Indians. The village dwellers, fearing that the cliff might fall on their house, attempted either to prop it up by a retaining wall or to further prevent erosion at the base. If they thought to prop up that great mass of rock with their puny masonry, they had an exaggerated idea of the quality of their work. If, on the other hand, as seems most likely, they intended merely to prevent wind erosion at the base of the cliff and thus forestall further undermining, their engineering ideas were far-sighted, sound, and effective. The rock retaining wall is now largely in ruins, but it has served for many centuries to prevent further erosion.

KAIBAB TRAIL BRIDGE AND TUNNEL AT SOUTH APPROACH
Grand Canyon National Park

Engineers will be interested to note that until 1887 Pueblo Bonito was the largest apartment house in the world. It consisted of approximately 800 rooms and housed more than 1,200 people. It was occupied for nearly 1,200 years, and was abandoned for some unknown reason approximately 1,000 years ago. Parts of the building now standing are four stories high, and much of the masonry in the lower sections is still in excellent condition.

PARK SANITATION COMPLETE

Regarding specific park improvements, it will be readily seen that in these unique areas it is impossible for the engineer to analyze the problems of improvement from the ordinary engineering viewpoint. Everything done or built must be viewed first from the point of view of its effect on the natural surroundings of the park.

For instance, most of the seven substations established in Yosemite Valley for the pumping of sewage have been sunk in the ground so that they are hardly visible to the tourist. Last year an activated sludge plant, designed by the U.S. Public Health Service, was completed in Yosemite Park at a cost of approximately $180,000. The plant has a capacity of 1,224,000 gal. per day and will take care of a valley population of 50,000 people. It is not an unusual plant, but unusual pains were taken, first, to hide the building, and second, to so construct it that it would not be unattractive.

In nearly all the parks, sanitation has been fully provided for by the installation of modern flush toilet systems and artistically built comfort stations. It is only in the outlying districts that the facilities are less adequate. In a few of the areas where water is extremely scarce, chemical toilet systems have been installed.

UNIQUE WATER PROBLEMS SOLVED

As might be expected, the best of water is usually obtainable in the National Parks. In the more frequented areas, all the campgrounds are provided with an adequate number of water faucets and hydrants conveniently placed. In some cases these systems have been rather extensive as compared with the amount of service rendered, but taken as a whole our water problems have been fairly simple.

On the other hand, in Mesa Verde National Park, dependence for water supply has been placed entirely upon rain. Two acres of metal roofs have been constructed to catch the rain, which is stored in reinforced concrete reservoirs having a total capacity of 700,000 gal. This supply has proved inadequate and wells have therefore been started. Already one well has reached a depth of 3,500 ft. Water is expected at a depth of approximately 4,200 ft.

TRAIL CONSTRUCTION ON THE WEST RIM
Zion National Park

At Hot Springs National Park, the Park Service has had a unique problem to solve, the conservation of the hot mineral waters for use in the many bath houses and hotels in that area. Water from the numerous hot springs was collected at the source, carried through insulated pipes to a main collecting pipe line, and thence into a collecting reservoir, all by gravity. Two reinforced concrete reservoirs, of 400,000 and 100,000 gal., respectively, were constructed high up on a mountainside. Electrically driven pumps elevate the water from the collecting reservoir to the storage reservoirs. All pipes are thoroughly insulated by being incased in cedar pipe with the proper air space between. Even the reinforced concrete reservoirs are insulated from the surrounding slopes by gridironing the foundations with tile, covering the entire foundation with coarse gravel, and placing a layer of crushed gravel around the outer sides of the reservoirs.

The hotel and bath-house industry at Hot Springs is dependent entirely on these springs; many millions of dollars have been invested by private individuals and companies. These interests were quite concerned as to whether the additional handling by pipe line and storage reservoirs would lower the temperature of the water to a point where it might have to be heated. As a matter of fact, it has been found that, because of the careful insulation, the temperature of the water delivered to the hotels and bath houses is approximately 5 deg. hotter than it used to be before the reservoirs were constructed, and when the old pipes, which were shorter but uninsulated, were in use.

The entire system is operated automatically by electrical devices. The construction of the lower pump house entailed the solution of an unusual problem; the blasting had to be done through seamy rock, which carried a constant flow of water at a temperature of 140 deg. The pump chamber is completely surrounded by hot water to a depth of approximately 6 ft.

ROADS AND TRAILS EXCAVATED TO LEAVE NO SCARS

The Park Service believes that any trail which is worth constructing is worth properly routing and locating. Formerly trails were located with a great rise and fall at every canyon. Waste of energy is just as

important to hikers and saddle parties using trails as waste of gasoline is to motorists using highways. Furthermore, by proper reconnaissance, scenic objectives are ascertained and the trails routed to reach them. In some cases the cost of properly located trails is no greater than that for those improperly located, but in general the expense is increased because of greater construction difficulties and the protective features used.

If, in constructing a trail through virgin country, heavy blasting were used to get rid of the rock—the cheapest and the usual method—the very object for which the trail was built would be defeated. The trail might reach certain objectives, but the scar on the landscape and the destruction below the trail would be so obtrusive for years to come, that few people would enjoy either a hike or a saddle trip over it.

ACTIVATED SLUDGE SEWAGE DISPOSAL PLANT
Yosemite National Park

MT. GOULD AND LAKE JOSEPHINE
Glacier National Park

In Yosemite Park on the Glacier Point Trail, which is less than five miles long, rises 3,000 ft., and has a ruling grade of 15 deg., great care was taken to blanket the shots, and the trail was so placed as to involve the least possible scar. Dry-wall and hand-placed rock embankments were used in many places to reduce the height of the scar and prevent the rock from flowing down the mountainside. The trail has even been oiled to prevent dust. Including all these features, the cost was $10,000 per mile.

In Zion National Park a footpath has been constructed leading through the Narrows of the Zion Canyon. Since this trail is used by practically all tourists, it was necessary that its surface should be dustless and smooth enough for the use of people coming direct from trains and ill-equipped for hiking. The trail blends harmoniously with its surroundings and reaches the desired geological, botanical, and other objectives, although at a cost far exceeding that of the ordinary hit-or-miss type of trail location and construction. The Park Service feels that such expenditures are entirely justified.

UNDERGROUND ELEVATOR RECENTLY COMPLETED

In the Carlsbad National Park there is a peculiar foot-traffic situation. The Park Service has built approximately seven miles of good underground trails in the caverns. Although the entire trail from the surface downward is interesting, the most fascinating section from a scenic standpoint begins at an elevation of 754 ft. below the surface. Since great numbers of people are incapable of lifting themselves 750 ft. to get to the surface, previously many lost the opportunity to see the underground wonders of the Carlsbad Cavern. They were thus accessible only to the physically fit. Now the Government has installed a passenger elevator system by constructing a shaft 7 by 14 ft. in cross section and 754 ft. deep. One elevator is now operating in this shaft, and a second is in process of being installed.

Excavation of the elevator shaft proceeded simultaneously from the top and bottom. Because of certain cavern controls, the lower end was first established and then the exact location above was determined by surveying an extremely accurate traverse through the caverns to the surface. It was of vital importance, not only that the central points of the upper and lower ends of the shaft should be exactly plumb, but also that the 7 by 14-ft. rectangle should be accurately oriented. The upper and lower headings met with an error of $\frac{1}{4}$ in.

Another unique undertaking was the lighting of these same caverns. The Government has installed two 120-hp. Diesel engines and generators, and is now putting in an additional 240-hp. Diesel generator. Seven miles of caverns have been lighted, and now this power is also being used to operate the elevators. All lighting in the caverns is indirect, so that in going through, the traveler is rarely aware of the lamps themselves but is fully conscious of the dazzling beauty of the various rooms and formations. The Park Service has taken advantage of the expert advice of the engineers of both the Westinghouse and General Electric companies, who have assisted its own electrical engineering staff.

SOLVING PROBLEMS OF BRIDGE DESIGN

The question of the proper type of bridge construction and the extent to which the Government is justified in spending money to make bridges ornamental is ever with us. It is definitely recognized that it is worth while, if possible, to utilize a type of architecture that will blend harmoniously with the landscape. On the other hand, it must be realized that a bridge will always look like a bridge no matter what attempts are made to blend it into the surroundings, or how much money is expended on it. This fact in many cases has been frankly conceded, and steel or concrete types have been adopted. Masonry arches are of course the most desirable for most locations in National Parks, where ruggedness of landscape is the rule.

Many excellent bridges were constructed in connection with the rebuilding of the road system in Yellowstone Park during the early years of the present century. A number of these are still in use, and two of them are deserving of particular mention, for they were at that time considered outstanding engineering achievements.

One of these structures is the concrete viaduct in Golden Gate, and the other is the arch bridge over the Yellowstone River. The latter, which was constructed in 1903, is situated about 1,000 ft. above the Upper Falls of the Yellowstone River near the Grand Canyon.

The span of the bridge is 120 ft. and the rise of the arch is 16 ft. The abutments are of natural rock. The arch contains ten steel girders, which were designed to carry, in addition to the ordinary dead load, the encasing concrete and a snow load equivalent to 4 ft. of solid ice.

CHITTENDEN BRIDGE, A HISTORIC STRUCTURE

The structure was called by its designer, the late Hiram M. Chittenden, M. Am. Soc. C.E., who had charge of the park from 1899 to 1906, a Milan-type arch. It would not be considered of any particular engineering interest if built today, but it was one of the first concrete arch bridges constructed in this country, and was designed at a time when comparatively little was known of the strength of reinforced concrete.

The work of supporting and building the falsework and pouring the concrete was quite an undertaking at the time. The bridge spans the stream at the swiftest part of the rapids, the Upper Falls of the Yellowstone being only a short distance below. The launching of the cribwork on which the temporary wooden pier was placed required considerable engineering skill. This crib was floated and held in place by means of heavy rope guides extending from both shores. Each half-span of the falsework was supported on five combination wood and iron trusses. The placing of the concrete is described by General Chittenden as follows:

> The concrete work, contrary to usual practice at that time with such large masses, was placed in one continuous operation. This would have been simple enough with a mechanical mixer, but was much more difficult where the mixing was done by hand. Elaborate preparations were made. All the material—gravel, broken rock, sand, cement—was gathered and distributed at convenient points about the site. A date was chosen in full moon in August and a temporary electric lighting plant was installed. A force of 150 men was assembled from the various road crews and divided into three shifts. When all was ready, the concrete mixing was begun and continued without intermission in 8-hour shifts for 74 hours, or until the work was complete.

Considering the fact that last year three million tourists entered the National Parks and Monuments, it is evident that the big problem of the parks is administration. The superintendent of each National Park is the key man, and nearly all the problems of that park revolve around him. Matters pertaining to education, engineering, landscaping, technical research, campgrounds, transportation, public facilities of all kinds, and contact with the public, are all integral parts of the administrative problem, which must be handled in a general way through the superintendent.

Park improvements cover a very wide range and are considered of vast importance by the Park administration. The engineering pertaining to them must be correlated thoroughly with all other branches of the Service. This is particularly true of the Landscape Architectural Division, to which we look for all architectural and landscape effects for all our projects. No building can be constructed, either by the Park Service or by the Public Operator, without the approval of the Landscape Architectural Division, both as to location and architecture. The director, Horace M. Albright, insists that there be the closest cooperation between the different divisions of the Service, and also with all the other governmental agencies that contribute to its success.

The National Park Service is operating for no other purpose than the dual one of serving the public and preserving the areas for future generations. We must and do remember that the chief value of the parks to the nation is in their inspirational and educational features, and that this fact must be recognized in making all improvements.

America's Park Highways

By L. I. HEWES
MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS
DEPUTY CHIEF ENGINEER, U.S. BUREAU OF PUBLIC ROADS
SAN FRANCISCO, CALIF.

SINCE 1924, the U.S. Bureau of Public Roads has been building for the National Park Service the roads in the National Parks and National Monuments, mostly in the 11 Western states. The bureau has also built a road in the Acadia National Park in Maine and now has under construction roads in the Shenandoah and Colonial parks, both in Virginia.

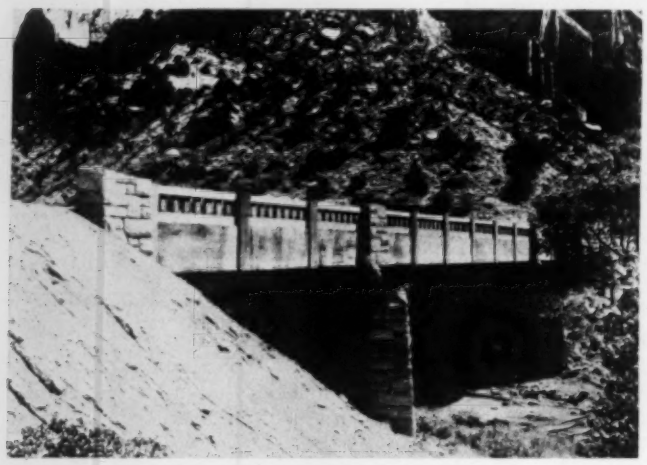
As at present laid out, this system includes roads of three types: those entering the parks as continuations of the Federal-aid or state systems; those located within the parks, such as branch roads to the tourist accommodation and hotel areas, usually planned for maximum scenic effect; and last, those that are primarily intra-park roads, for the development of the recreational potentialities of these areas. Most of the roads serve more than one purpose: for example, nearly every mile is a scenic highway. More recently, some of the park appropriations have been made exclusively for the construction of approach roads outside the limits of the parks. These roads, which must lie 90 per cent on Government-owned land, constitute essential links between the park systems and the Federal-aid and state highways outside.

HAPPY ISLES BRIDGE OVER THE MERCED RIVER
Yosemite National Park

Naturally, the first objective has been to complete the extensions of Federal-aid and state roads into the parks, and this phase of the work is well advanced. Priority of construction has been determined largely by the existing traffic. These extensions of main arteries of

traffic into and through the parks in many instances have formed essential additions to the Federal-aid system.

Typical of the roads having as their main purpose the development of the potentialities of the parks themselves are the proposed Rim Road around Crater Lake, the Cape Royal Road on the north rim of the Grand Canyon,



VIRGIN RIVER BRIDGE, BUILT IN 1930
Zion National Park

the Glacier Point Road in Yosemite Valley, and the Bear Lake Road in Rocky Mountain Park, Colorado.

MUCH HEAVY EXCAVATION REQUIRED

All the National Parks and some of the National Monuments are in extremely rugged terrain, and thus much of the construction work has been of the heavy mountain type. There are, however, notable exceptions. For example, the 50-odd miles of road between Williams, Ariz., and the south boundary of Grand Canyon National Park are in flat mesa country. The roads on the south rim of the Grand Canyon are through slightly rolling country, as are also most of those on the north rim. On many miles of road in Yellowstone Park, construction was apparently easy, but in some places it involved very heavy work, notably at the east entrance, through Sylvan Pass from Cody.

In building these roads a fairly high standard of cross section, alignment, and grade has been adopted. Surfacing has been extended to a minimum width of 18 ft., and many of the roads are surfaced to a width of 20 ft. The sections in cut are generally 27 ft. wide over all. Ruling grades are 5 per cent, although there are many stretches where a 6 per cent grade was used, and a few difficult places where a 7 per cent grade was required. Curvature, which from the operating standpoint is the most important element of highway design, has been given particularly careful attention. Every road and almost every curve have been specially designed. For example, in Zion National Park, a succession of six switchback curves were built with radii ranging from 90 ft. to 150 ft. All these turns were so compensated, widened, and protected with guard walls that they are not a hazard to the driver.

It has been recognized that the purpose of park highways is recreation—that they are not primarily thoroughfares. Nevertheless, all of them must be safe under any conditions of peak load or fast traffic that are likely to develop. In many cases, as in the Crater Lake, Rainier, Sequoia, Lassen, and Yosemite parks, and particularly in the Rocky Mountain Park in Colorado, it has been necessary to reach high elevations with long sustained grades. In locating such roads, the snow problem is ever present; therefore the choice of exposure is important. Wherever possible, south exposures are utilized, but the problem is somewhat complicated by the requirement of attaining certain scenic objectives. In some instances the terrain has not permitted the choice of a south exposure. Always it has been recognized that the park highways now under construction are in their final locations.

ADVICE ON LANDSCAPING FREELY USED

In all the work in the National Parks, the Bureau of Public Roads has been guided in its design by the Landscape Division of the National Park Service. Landscaping of highways is comparatively new in America, and the bureau has been fortunate indeed in having the effective cooperation of a splend.d group of landscape architects. The landscaping of the National Park Highway System has as its essential aims the diminution of scars; the introduction of certain elements of grace in alignment; the use of architecturally pleasing structures; and the protection of trees, shrubs, and other natural growths from destruction and damage during construction. Although it is not possible to eliminate all scars in cuts or fills on mountain highways, it is possible to reduce them greatly in number and size. The reduction is accomplished by the judicious use of riprapped embankments and, less frequently, by the building of retaining walls and careful design of the grade to permit end-hauling to invisible gullies. Naturally such work is more expensive than the hit-or-miss type of grading more generally employed outside the parks. It is possible to reduce overbreak in rock by careful shooting, and also to prevent the scattering of such material below the toe of a slope.

The influence of the park landscaping methods has been felt by the bureau's entire organization and has resulted in better looking roads outside the parks. No longer is it permitted to waste material in unsightly places on the National Forest Highways. The highway departments of the Western states have also been influenced by the results achieved on the park roads, and there has been a noticeable improvement during the last few years in the appearance of the roads constructed by the states themselves.

In the matter of curvature, operating safety is never sacrificed to landscaping effects. In general, curves have been designed to fit the topography as closely as

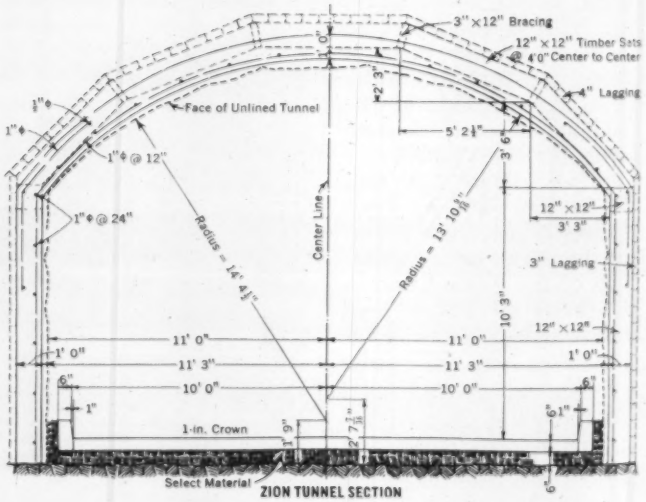


FIG. 1. SECTION OF ZION TUNNEL, IN ZION NATIONAL PARK

possible without sacrificing sight distance. There has been a marked tendency to eliminate "broken-back" curves by introducing long, carefully compounded curves with gradual change in length of radii. All the work in the parks has been sufficiently recent to take advantage of the most advanced ideas with respect to super-elevation and widening on curves. All the curves have grade compensation, which is increased when the ruling grade is 5 per cent or more.

Throughout the National Park system, at points of special scenic interest, turnout and parking places have been provided and protected by special stone masonry parapets or guard walls.

The average cost for the grading and surfacing of roads in the parks has been approximately $40,000 per mile. All kinds of surfacing have been used, but it is planned that all the park roads will be provided with dustless surfaces. For the most part, some form of bituminous top will be used. Where conditions are favorable, road mix may be resorted to, utilizing light oil or cut-back asphalt. The minimum width will be 20 ft.

Such surfaces are usually built of fine crushed material at least 3 in. thick, laid on a base consisting of 4 in. of fine crushed stone rolled into place. The great advantages of a light oiled surface are its excellent riding qualities and the ease with which it can be scarified and relaid in case of trouble. It is too soon to predict the ultimate life of such surfaces.

It has recently been decided somewhat to modify the type of roadway section developed and used by the Bureau of Public Roads in the Western mountains during the past decade. The objective is to eliminate the relatively steep though narrow ditch and to substitute a broad shallow type, which will be much safer for traffic and serve to increase the total driving width of the highway. It will be necessary to use tile underdrains with suitable backfill in some sections of a road of this type, and paved gutters in others. Another new feature will be the extension of the crushed gravel surfacing entirely across the road bed. It is felt that the added expense will be amply justified by improved operating conditions, including better snow removal and cheaper maintenance.

Considerations of landscaping have been especially important in bridge and culvert construction. Very few steel or straight concrete structures have been used. For the most part, arch bridges with concrete barrels and natural stone masonry facing have been built. Wherever possible, culverts are of stone, but if pipe culverts must be used, they are provided with stone masonry head walls. With few exceptions, guard walls

WEST PORTAL OF THE ZION TUNNEL
Tunnel Is 5,613 Ft. Long; Roadway, 20 Ft. Wide

are built of cement rubble masonry, according to a carefully chosen design. In recent years, mostly to produce suitable architectural effects, the use of arch bridge designs has been extended to crossings for which they would hitherto have been considered inadvisable. Even sites formerly considered entirely impossible for this type of bridge have been ingeniously utilized. The bridges on the recently finished Mount Vernon Memorial Highway are noteworthy examples. Several of these arch masonry structures are founded on nothing but silt. Similar conditions have been met in the National Parks.

It is not, however, always possible to build arch bridges at a reasonable cost. The bureau has adopted engineering designs to fit architectural requirements of great variety. Over the Rio Puerco in Arizona a bridge has been recently finished consisting of a series of steel spans on concrete piers. The steel girders are shaped to resemble arches. The specially designed concrete piers of this bridge form a pleasing element in the design. In Zion National Park, over the Virgin River, a deck bridge was constructed of deep steel I-beams resting on heavy masonry piers, in which the I-beams were completely concealed by a wooden facing.

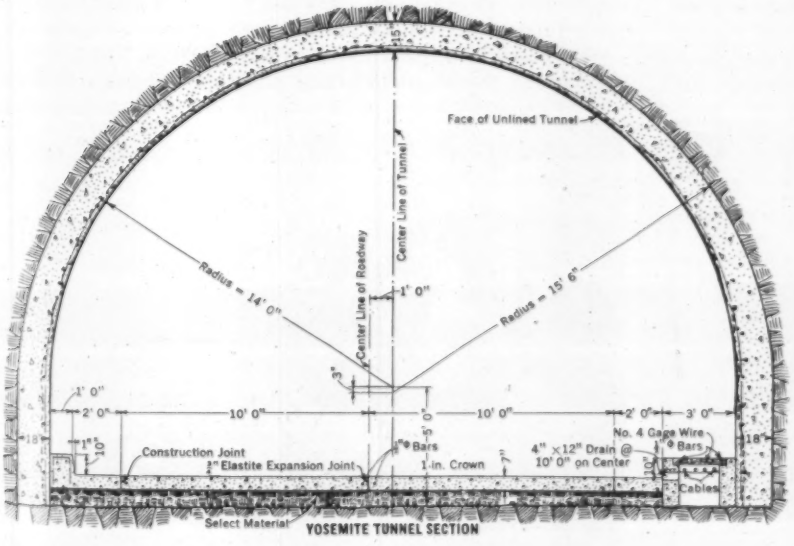


FIG. 2. SECTION OF THE WAWONA TUNNEL
Yosemite National Park

TWO NOTABLE HIGHWAY TUNNELS

Two notable highway tunnels have been constructed, one in Zion National Park and one in Yosemite. The former, cut through cliffs of pure sandstone, is 5,700 ft. in length on a 5 per cent grade. The center line of this tunnel intersects the base of the cliff at six points where its direction changes. At these points galleries or great windows have been opened into Pine Creek Canyon, which the tunnel roughly parallels, producing interesting and pleasing scenic effects. The largest gallery is more than 80 ft. long, approximately 30 ft. high above the pavement, and nearly 40 ft. wide. On ac-

count of the broken character of the ceiling rock, part of the Zion Tunnel was lined with timber, particularly where the strata become thin as the grade rises. The remainder was protected with a gunite arch about one inch thick. Where the exposed rock walls show indication of alteration and where increased hair cracks are evident in the gunite crown, a full concrete lining will be built and the remainder of the vertical walls will be covered with gunite down to the road grade. This tunnel is 22 ft. wide in cross section (Fig. 1) and has a maximum vertical clearance of 14 ft., measured from the crown of the road to the semicircular top. The tunnel is paved with concrete and provided with two 6-in. curbs. Under present traffic conditions, no lights other than the headlights of the passing cars seem to be needed. No congestion has yet been observed at any of the galleries, although cars frequently stop for observation. The contract price for driving the Zion tunnel was $55 per lin. ft.

FROM THE TOP OF MT. OBERLIN
Trails Built by Contractor to Reach His Work

The second tunnel, in Yosemite National Park, is now being completed between Turtle Back Dome and Bridal Veil Falls. It is known as the Wawona Tunnel and is 4,320 ft. long. It was driven to avoid cutting a severe and permanent scar in the south granite escarpment of the valley. Various alternate positions of the road and modifications in location were considered before it was determined that a tunnel would be necessary. It was holed through in January of this year at a contract price of $70 per lin. ft.

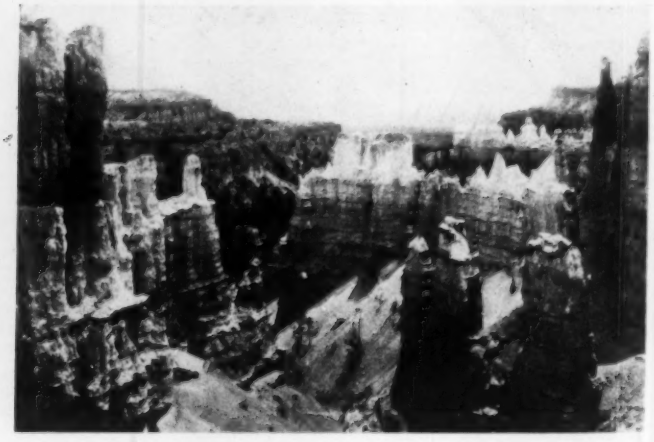
As the holiday traffic will be large, the cross section, shown in Fig. 2, is larger than that of the Zion Tunnel. The over-all width is 28 ft. and the height to the crown is 18 ft. The tunnel will be paved with concrete, and provided with concrete curbs on both sides and a 4-ft. sidewalk on one side. The alignment is straight and on a 5 per cent grade throughout. The tunnel was driven through granite, entirely from the east end, as all the muck had to move downhill into the road embankment between Bridal Veil Junction and the East Portal. About 750 ft. of the tunnel must be concrete lined. Tunnels for highway traffic must be completely protected against any falling rock whatever; consequently sections that might be tolerated in a railroad tunnel require lining for highway operation.

An interesting feature of the Wawona Tunnel is the provision for ventilation by three horizontal adits driven at the quarter points of the tunnel's length. The middle adit, 500 ft. long, will be of full tunnel section. It will be supplied with ventilating fans arranged to run either to exhaust the air from the tunnel or to supply fresh air from beyond the face of the cliff. The operation of the fans will be controlled by automatic carbon monoxide detectors. The other two adits are 14 by 14 ft. in cross section and serve as escapes in case of fire and for additional ventilation. The ventilation facilities were designed in cooperation with the U.S. Bureau of Mines, whose recommendations have been followed throughout. It is expected that this tunnel will be finished and in operation in time for the beginning of the 1933 park season.

TWENTY MEN MOVING COMPRESSOR
Along Garden Trail
Logan Pass

All the roads constructed by the U.S. Bureau of Public Roads for the National Park Service have been laid out according to a carefully planned system, including about 1,800 miles. Approximately $25,000,000 has already been spent to complete about one-third of that mileage.

ENJOYING RAINIER NATIONAL PARK
Summit of Mountain Is Only Six Miles from the Camera



BRYCE CANYON NATIONAL PARK
Looking Down Into the "Sculptor's Studio"

Rise and Fall of the Public Domain

Discussion of Policies Involving a Billion and a Half Acres in the United States Proper

By HERMAN STABLER

MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS

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AT THEIR maximum, the public lands in the continental United States amounted to the amazing total of almost one and a half billion acres, equivalent to about half the total area of the country. According to the early point of view, the public domain was merely an asset to be liquidated as quickly as possible for the financial benefit of the nation. Since 1841, the public lands have been used chiefly as areas for settlement and as a means of upbuilding the nation. Under this program much of the territory—almost a billion acres—was disposed of. The manner in which the public domain was acquired, methods of disposal, present problems of control, and particularly advisable future policies, are treated here by a Government official charged with large responsibilities in connection with the public lands. This paper has been abstracted from Mr. Stabler's more complete treatment presented on July 6, 1932, before the general technical session at the Society's Convention in Yellowstone National Park. Credit for much of the material used is due to G. W. Holland and J. F. Deeds, of the Conservation Branch, U.S. Geological Survey.

THE story of the public domain, like the story of the United States of America, begins with the signing of the Declaration of Independence on July 4, 1776, which was followed promptly by cession to the new Federal Government of 260,000,000 acres of public lands (Fig. 1), an area half again as large as the public domain remaining unreserved and unappropriated in 1932.

The states of Ohio, Indiana, Illinois, Michigan, Wisconsin, Tennessee, and parts of Minnesota, Alabama, and Mississippi were carved out of this original cession. The Louisiana Purchase in 1803, the Florida Purchase of 1819, the Mexican cessions of 1848 and 1853, and the Texas cession of 1850, added 1,200,000,000 acres to the public domain and rounded out the vast public estate of a billion and a half acres acquired for an aggregate purchase price of $81,000,000, or a little less than 5½ cents an acre. Subsequent acquisition of Alaska and other outlying possessions—which are not considered in this article—added another half billion acres to the territorial jurisdiction of the United States.

Two-thirds of the Federal landed estate has now passed into private ownership, as shown in Fig. 2. Disposal of it began as soon as it was acquired. In August 1776, the Continental Congress, in answer to an invitation by Great Britain for our troops to desert, offered each foreigner who would leave the British armies an estate from the public domain—1,800 acres to a colonel, 600 acres to a major, and so on down to 50 acres to a private.

For nine years disposal was made by direct grant. Sales primarily for revenue were made for nearly two-thirds of a century. Settlement became the dominant idea with the Preemption Act in 1841, which was followed by the Homestead Law in 1862. More than a fifth of the land disposed of has been granted to new states in aid of schools, transportation, and reclamation; more than two-thirds has been granted to individuals, mostly for purposes of settlement and the upbuilding of the nation; and less than 10 per cent has been granted to corporations, mostly in aid of transcontinental transportation systems.

The remaining Federal estate (Fig. 3) consists of less than half a billion acres—less than a third of the original area.

In general, the reserved public lands are productive of revenue; the parks and monuments yield about $1,000,000 and the forests about $5,000,000, amounts much less, however, than the annual appropriations for the park and forest services. The reserved mineral deposits are of outstanding interest because, under the leasing system now existing, they bring in revenue about 20 times all costs of classification and administration.

Production of mineral fuels, the most important leased mineral deposits, is shown in Fig. 4 by fiscal years, and in common terms of equivalence, that is, in millions of tons of coal. Mineral fuels equivalent to about 11,000,000 tons of coal are produced annually from leases under supervision as to methods of prospecting and production with a view to conservation of life, health, and property.

The revenue from mineral leasing (Fig. 5) is normally between $5,000,000 and $10,000,000 a year, but now, on account of overproduction and depressed prices, is little more than $3,500,000. The chart shows clearly the effect on revenues of overproduction in oil, gas, and gasoline, beginning with the development of the Seminole fields of Oklahoma in 1927. Only 10 per cent of the revenue indicated goes to the

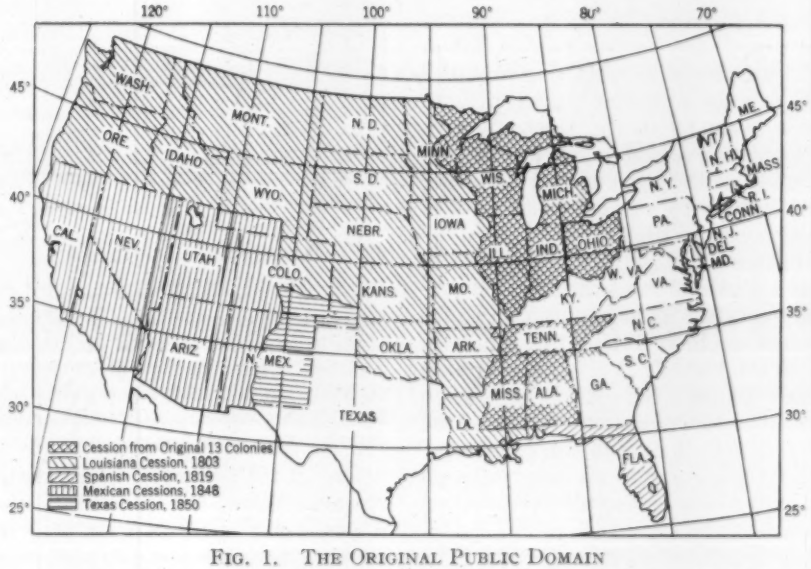


FIG. 1. THE ORIGINAL PUBLIC DOMAIN

From the Report of the Public Land Commission Dated January 18, 1881

Federal treasury. The states within which it is derived, chiefly California and Wyoming, receive 37½ per cent directly; and 16 Western states receive 52½ per cent indirectly, through expenditures for reclamation projects within their borders.

Existing legislation provides in a fairly satisfactory manner for the effective administration of most reserved lands and rights. The remaining problems of the public domain relate chiefly to unreserved lands and include cadastral surveys, classification, protection from fire and trespass, watershed protection, conservation and use of forage resources, and ultimate disposal.

Scientific classification is an essential prerequisite to the lease or disposal of lands. Any intelligent landlord will find out for what his property is valuable prior to disposing of that value by permit, lease, or sale. So the United States authorized classification of the public lands in 1879 and in recent years has made small annual appropriations for that purpose.

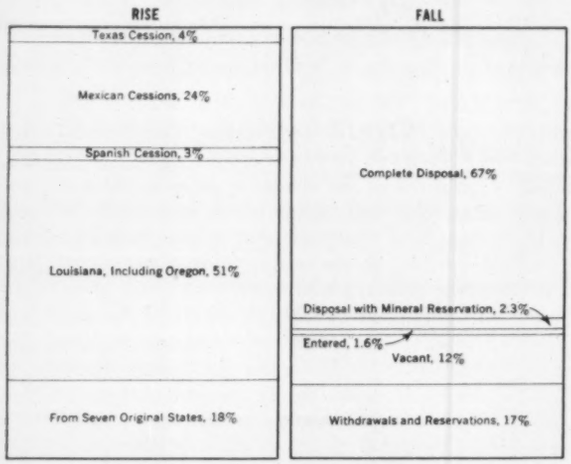


FIG. 2. RISE AND FALL OF THE PUBLIC DOMAIN
1½ Billion Acres Reduced to About One-Third in 1932

Classification of public coal lands (Fig. 6) was initiated in 1906 and, until the World War, proceeded at a rate that would have provided for completion of the task about ten years ago. The appropriations for, and progress of, classification were then slowed down until it appears to be a never-ending project. "Economy" in the fiscal year 1933 will put a substantial stop, temporarily at least, to further progress.

Progress in water-power classification, as measured by river surveys (Fig. 7), has suffered a similar decline in recent years and bids fair to require a hundred years or more for what might reasonably be a twenty-year job.

Agricultural and grazing classification was first appropriated for in 1917, and fairly satisfactory progress is being made, as evidenced by Fig. 8, which shows the areas covered by classification maps published or in process of publication.

Delay in classification is costly. The great Kettleman Hills oil field of California provides an outstanding example. As a result of such delay in this case a billion dollars worth of oil land that should be returning royalties of more than $100,000 a month is now under private ownership, which affords a barrier to the most effective unit operation. But Congress seems to prefer cutting some thousands of dollars from the current year's meager appropriation for classification to saving vast sums in mineral values to the taxpayers of the nation.

"Protection of watersheds" is a term, apparently coined by foresters, that appears in recent legislation pertaining to the National Forests. It is understood to relate primarily, though not exclusively, to the influence of vegetation—trees, grass, weeds, or whatever—on stream flow and erosion. Though the forest reservation acts limit the character of lands to be reserved to those "wholly or in part covered with timber or undergrowth" many additions to National Forests of non-timbered lands have been urged and not a few made on assumed value for "watershed protection."

Erosion on parts of the public domain is heavy. The Colorado River basin, particularly that section in southeastern Utah, northeastern Arizona, southwestern Colorado, and northwestern New Mexico, affords an outstanding example of heavy uncontrollable erosion on the public domain. The annual contribution to the Colorado River from 60,000 square miles in this region amounts to about 21 acre-ft. of water and 3,000 tons (1⅔ acre-ft.) of suspended matter per square mile of drainage basin. It effects denudation at the rate of about 0.03 in. per year. Less than 10 per cent of the water and more than 70 per cent of the sediment that reaches the Grand Canyon is derived from this region, which is characteristically a barren plateau country without a vegetational aspect. The precipitation, averaging less than 10 in. per annum, comes chiefly in torrential summer showers, which, falling on bare weathered areas of soft sandstone or friable shale, produce veritable rivers of flowing sand and mud.

The phenomena of such a shower in this area observed by C. E. Erdmann, geologist of the U.S. Geological Survey, may be described as follows:

> About 3:30 p.m. on August 6, 1926, a storm was noted near the headwaters of Hunter Creek, Mesa County, Colorado, about ten miles away. Shortly after 4 p.m. the skies then being clear, a dull roar like a distant waterfall was heard and grew rapidly louder, advance notice of an approaching flood. About 4:30 p.m. the flood came down the dry channel of Hunter Wash pushing before it a heavy mat of dead timber, brush, and rubbish. The roar of the stream was deafening, the ground trembled noticeably, and at intervals there would be a hollow, resonant impact as boulders of the traction load crashed into one another. The depth of the flood was about 7 ft.; width, 25 to 30 ft.; velocity, 10 to 12 miles an hour; crest height, reached in a few minutes and held for 15 to 20 min.; duration, 2 hours; estimated volume, about 150 acre-ft.; suspended load, estimated from bucket sample, 30 acre-ft.

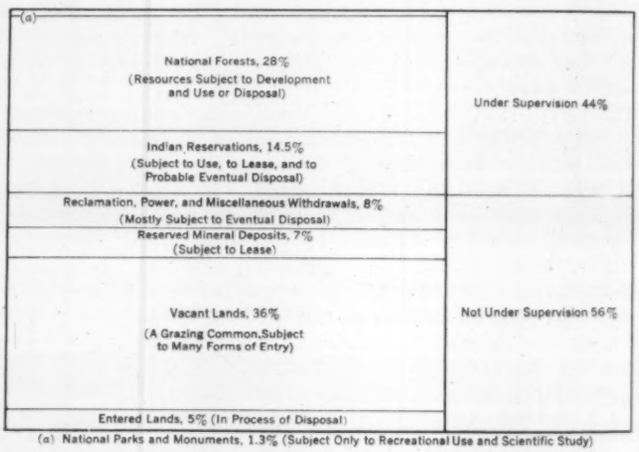

(a)
National Forests, 28%
(Resources Subject to Development and Use or Disposal)
Indian Reservations, 14.5%
(Subject to Use, to Lease, and to Probable Eventual Disposal)
Reclamation, Power, and Miscellaneous Withdrawals, 8%
(Mostly Subject to Eventual Disposal)
Reserved Mineral Deposits, 7%
(Subject to Lease)
Vacant Lands, 36%
(A Grazing Common, Subject to Many Forms of Entry)
Entered Lands, 5% (In Process of Disposal)
Under Supervision 44%
Not Under Supervision 56%
(a) National Parks and Monuments, 1.3% (Subject Only to Recreational Use and Scientific Study)

FIG. 3. THE REMAINING FEDERAL ESTATE
Half a Billion Acres in 1932

One of the photographs shows the erosive action in the stream channel.

One such characteristic little storm a year for each seven square miles would account for the water and sediment contributed to the Colorado River from the entire

"four-corners" region, an area as large as New England. Protection of a watershed against the effects of such storms by fostering vegetable growth is wholly impracticable.

Erosion is a fundamental process of land sculpture that has been continuous through the ages, with general resultant benefit to mankind. To it may be attributed the fertile valleys that are the mainstay of agricultural development and, in contrast, the scenic wonders of the hills and mountains. Locally and temporarily, floods and erosion cause inconvenience and expense. It is an open question, even on a local and temporary outlook, whether erosion on the remaining public domain exerts an influence mainly detrimental or mainly beneficial to human activities. Widespread efforts to control erosion there would involve costs out of all proportion to the benefits derived. On the other hand, normal range management may be expected to have a tendency toward such limitation of erosion on the public domain as is practicable. It is evident that "watershed protection," *per se*, is not an important problem affecting the public domain.

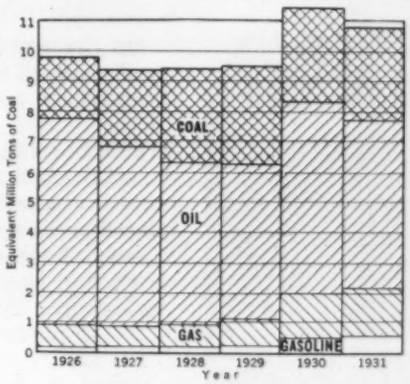


FIG. 4. FUEL PRODUCTION ON THE PUBLIC DOMAIN

1 Ton of Coal = 4½ Barrels of Oil = 208 Gal. of Gasoline = 26,000 Cu. Ft. of Gas

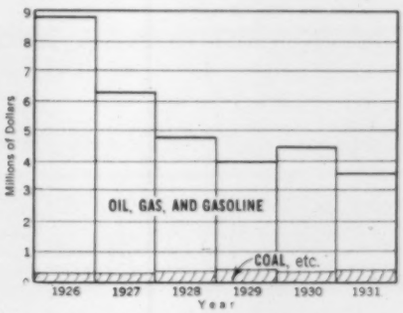
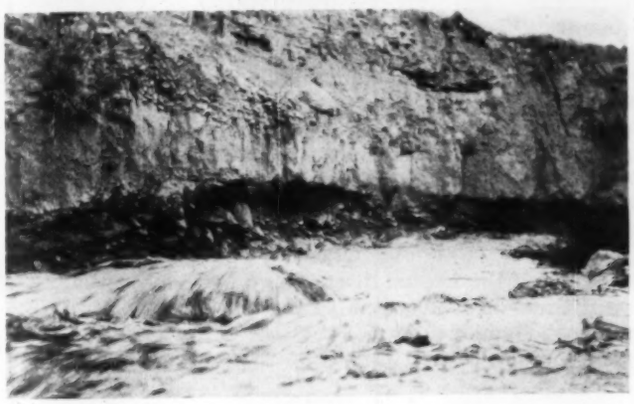


FIG. 5. REVENUE FROM LEASES OF MINERALS ON THE PUBLIC DOMAIN BY FISCAL YEARS

GRAZING AN IMPORTANT FACTOR

Conservation of agricultural resources on the public domain has been accomplished mainly by laws to encourage development by individual effort and concurrent transfer from public to private ownership. The agricultural land laws substantially ignore the use of land merely for grazing purposes, and there now remains on the unreserved public domain very little land of agricultural value other than grazing. The outstanding agricultural problem of the unreserved public domain is conservation of forage resources.



UNDERCUTTING OF A BANK ON HUNTER CREEK, COLO. Caused by a Cloudburst Upstream

About 99 per cent of the vacant unreserved public domain lies in the 11 states of Arizona, California, Colorado, Idaho, Montana, Nevada, New Mexico, Oregon, Utah, Washington, and Wyoming, and about 90 per cent lies between the Sierra Nevadas and the Rocky Mountains and between the latitude of the northern boundary of Wyoming and the Mexican border. In general, the only agricultural use or value of this land is for grazing, and most of it is useful for this purpose only during limited seasonal periods.

The forage resources of the unreserved public domain have an average annual income-producing value to the livestock industry of about $35 per square mile and a probable average annual leasing value, measured in terms of normal leasing charges, of about $9 per square mile. Obviously the expenditure of $800 a square mile in improvements ($1.25 to the acre), as required under the stock-raising homestead act, to obtain title to land that can at best produce an income of about 4 per cent of that amount, is poor economics. The average vacant public land will not earn an income from grazing sufficient to justify private ownership except as an adjunct to more valuable holdings and under a very low tax valuation.

REGARDING RANGE REGULATION

The traditional Federal policy has been to regard grazing on the unreserved public domain as a right for the free and unrestricted use of all alike, and a similar policy has been followed on reserved lands except where unregulated grazing interfered with the purposes of the reserve. Originally this policy was beneficial because the supply of range feed far exceeded the demand and the free grazing furnished an inviting prospect for stock growers, who blazed a trail beyond the frontier of settlement for the later development of more intensive agriculture. On the public domain today, however, the demand for range feed is far in excess of the available supply. The policy of unrestricted use not only is no longer beneficial but is seriously harmful because it results in impairment in the value of the range through overgrazing.

The need for a policy of range regulation, or rather, apportionment of range privileges, was recognized in pioneer days. The draft of a bill creating "pasturage districts" was included in a report by J. W. Powell on the arid lands in 1878, and the Department of the Interior ever since has urged upon Congress the need for a grazing law. Congressional committees and other Government agencies, as well as private associations, have also urged the need for grazing legislation, but all alike have failed to devise a formula acceptable to the divergent interests involved. Congress now has under consideration a plan of grazing control endorsed by the Departments of the Interior and of Agriculture that adds to the present public domain administrative functions, authority to regulate grazing on that domain.

THE DOMAIN AS A WHOLE

The public domain in 1932 consists of a little less than half a billion acres of land (Fig. 3), less than 40 per cent of which is unreserved and unappropriated. Among its assets should be listed extensive recreational areas, a

substantial proportion of the nation's timber supply, fuel resources sufficient for thousands of years, miscellaneous mineral resources of vast quantity in the aggregate, more than a third of the nation's water power, most of the remaining irrigation resources of the arid region, and forage resources sufficient to feed millions of stock. The surface of the lands reserved as National Parks, Monuments, and Forests, and of Indian Reservations, is under Federal administrative supervision, and the resources of the forests and Indian Reservations are available for use under regulation. The water resources and mineral resources are either being developed by the Federal Government or are subject to lease for development by private parties under Government regulation and supervision.

BADLANDS OF FRIABLE SHALE NEAR CAINVILLE, UTAH
From Which Comes Much of the Silt in the Colorado River

EXPENDITURES BY GOVERNMENT GREATLY EXCEED INCOME FROM PUBLIC LANDS

Only the surface of the unreserved, unappropriated lands is open to unrestricted use and abuse without some sort of interested care or regulation. For the fiscal year 1932, revenues from the public domain totaled $17,000,000, of which nearly $14,000,000 was designated by law for payment to, or for the benefit of, the public-land states; but the expenditures of the Federal Government by reason of the public domain exceeded $60,000,000. The Federal public domain is an expensive luxury. Its budget is far from balanced. What should be done about it?

It has been a long-settled principle of national law that, cost what it may, the Federal Government shall retain ownership of National Parks, Monuments, and Forests and the nationally owned mineral and water-power resources. Of late years this doctrine has been expanded and millions of dollars of Federal money have been spent to purchase additional lands to be included in National Parks and Forests.

GARFIELD COMMITTEE RECOMMENDS POLICIES

On January 16, 1931, a committee of 20, appointed by the President, with James R. Garfield as chairman, made a report on the conservation and administration of the public domain. In substance the report recommends: (1) that existing reservations for public purposes should be maintained and extended and that "all portions of the unreserved and unappropriated public domain should be placed under responsible administration for the conservation and beneficial use of its resources"; (2) that private ownership should be the final objective in the case of agricultural and grazing lands not "advisable or necessary for public use," but that the states within which lands chiefly valuable for the production of forage are situated should be granted a ten-year option for acceptance of such lands as a conditional gift if they "can be effectively conserved and administered" by said states; and (3) that when any state shall have developed and accepted a conservation program for mineral resources based "upon such uniformity of Federal and state legislation and administration as will safeguard the accepted principles of conservation and the reclamation fund," the mineral resources within its border should likewise be transferred to that state.

The Garfield report has been widely hailed as an anti-conservation document on the theory that its recommendations, if adopted, would turn the nation's remaining resources over to the states for exploitation by local politicians and greedy corporations. It has been roundly "booed" by state officials and others because it proposes to give to the states only the least valuable resources and to retain under Federal ownership, for the time being at least, the more valuable lands of the public domain; and the opportunity has been seized upon to renew with vigor the century-old campaign to turn over to the states all the reserved as well as the unreserved Federal estate within their borders.

THE SOUND VIEW OF THE PUBLIC ESTATE

Much of the heated discussion on the subject is based on a total misconception of the public domain and its value. There is no doubt that the entire Federal landed estate, with its vast timber, mineral, water, and agricul-

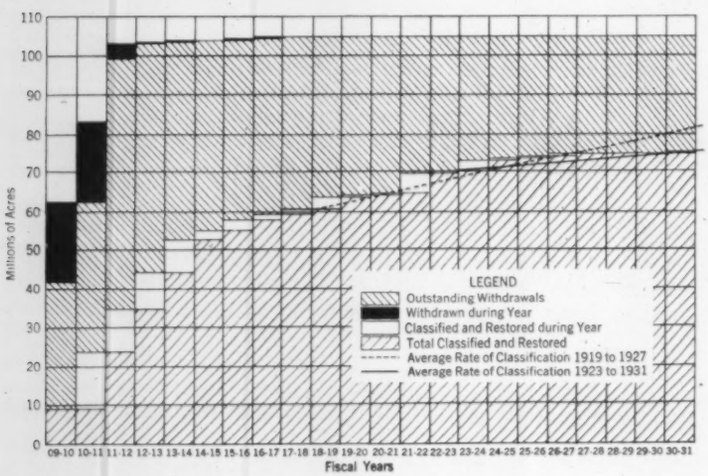


FIG. 6. PROGRESS OF CLASSIFICATION OF COAL LANDS, 1909–1931

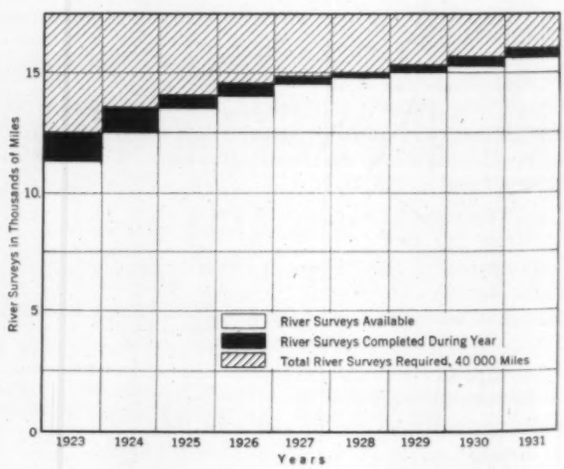


FIG. 7. PROGRESS IN THE MAKING OF RIVER SURVEYS

tural resources, is of inestimable worth in the life of a nation, if its essential values are conserved and its resources developed and used with due regard to economic needs; but it should be clearly recognized that its present worth, measured in terms of capitalized current gross revenue, is far under a dollar an acre, and that the cost of administration and supervision is several times the income that can reasonably be derived. The alleged greedy corporations cannot afford to own it. Local politicians can find little profit in it. The taxpayer, Federal or state, must bear the burden of current administration.

What if the public domain does contain two hundred billion tons of coal? Coal production on the public domain amounts only to about 3,000,000 tons a year. Coal to be mined a century hence has only potential value—value to the nation if conserved—but no present worth to the individual or corporation. Oil and gas alone seem outstanding profit-making possibilities, though some other resources could be made to pay their way.

GAINS AND LOSSES TO STATES

On the whole, the task of caring for and disposing of the remaining public domain is impressed with the same duties and responsibilities and the same lack of possibilities for immediate profit that has characterized ownership of the greater public domain from its inception in the eighteenth century. It is characteristically a task for the nation. All the states will benefit if the resources are wisely developed and used and not subjected to untimely exploitation. The public-land states will benefit, in addition, and in various ratios by reason of activity engendered by development within their borders.

Under present rules of Federal ownership and control, 37½ per cent of the gross income from mineral, and 25 per cent of the income from forest, resources of the public domain is paid to the states from which derived, while 52½ per cent of public-land sales and mineral receipts is distributed for irrigation development in 16 Western states. There are 32 states which, by contributing to the net cost of public-domain activities, are, in effect, granting to the remaining 16 subventions from which the former receive benefits only in the general upbuilding of the nation. Were the public lands and their resources to be granted to the states within which they are situated this would all be changed. The outstanding oil and gas states, California and Wyoming, would derive immediate profit from the change, but all others would suffer. In the course of centuries the outstanding coal states, Colorado, Montana, New Mexico, North Dakota, Utah, and Wyoming, might benefit relatively at the expense of the other public-land states, though it is questionable whether the loss of Federal subvention would be offset by any possible gain.

SEMI-DESERT VEGETATION
Similar to That on More Than Half the Unreserved Public Domain

It is believed that the Garfield report: (1) is sound in its endorsement of continuation of the existing policy of Federal reservations for public purposes; (2) should receive approbation for its proposal that all portions of the unreserved and unappropriated public domain should be placed under responsible administration or regulation for the conservation and beneficial use of its resources; (3) may well be sanctioned in its conclusion that private ownership of public agricultural and grazing lands should be an objective of final disposition; and (4) reflects the public interest in its suggestion that any grant to the states of lands chiefly valuable for the production of forage should be predicated on effective conservation and administration by the states containing them, and that any grant to the states of mineral resources should be predicated on the wise use of such resources under a program based on uniformity of state and Federal legislation and administration such as to safeguard their use in conformity with accepted principles.

In conclusion:

Whatever policy of ownership and control will contribute most effectively to the conservation and eventual wise use of the vast natural resources of the Federal estate is the one that will best serve the public interest.

Existing Federal policies and practices, except as to grazing lands, though imperfect in detail, are in general wisely founded and reasonably adequate.

Legislation permitting extension to all grazing lands of the types of administration already proved effective and satisfactory on some of them is highly desirable.

Federal ownership and control should be relinquished only on a clear showing of more effective and wiser conservation and use under other auspices.

FIG. 8. PROGRESS OF AERIAL SURVEYS
Of Agricultural and Grazing Lands

FIG. 9. PUBLIC RANGE LANDS
Still in Government Possession

Duty of Water in Terms of Canal Capacity

Stressing the Need for a Joint Engineering, Agricultural, and Economic Study

By E. B. DEBLER

MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS

HYDRAULIC ENGINEER, BUREAU OF RECLAMATION, DENVER

MOST of our irrigation systems have evolved from small beginnings by the gradual enlargement and extension of canals and provision of additional storage. The remaining new projects generally call for a type of canal construction incapable of economical enlargement and requiring determination in advance of the ultimate needed capacity. Therefore careful consideration must be given to agricultural, economic, and engineering features, as here enumerated in detail. The Irrigation Division's Committee on the Duty of Water in Terms of Canal Capacity, of which Mr. Debler is chairman, is endeavoring to determine the amount of canal losses and their effect on capacity. This progress report of the committee was presented on July 7, 1932, before the Irrigation Division at the Annual Convention of the Society in Yellowstone Park.

EARLY irrigators in this country were practical if not exact in their methods. Ditches were dug by hand, the water following behind the workman to confirm his idea of levels, or more often a line was laid by means of a carpenter's level on the standard grade of an inch to the rod, and plowed out with a team. Such ditches were enlarged and extended from time to time in accordance with the irrigator's practices or the available water supply.

After a time, entire irrigation projects were planned before construction started and often carried out in one operation. This made it necessary to determine canal capacities in advance of construction, particularly where controlling features such as tunnels, siphons, and flumes were involved. Many extensive projects were launched and constructed with a remarkably small amount of engineering planning. That they succeeded as well as they did is a great tribute to the common sense of the planners, few of whom would have claimed the title of irrigation engineer. The short space of half a lifetime spanned the period from the beginning of irrigation in this country to the development of single projects serving a thousand or more farmers.

The engineers who planned these projects were drawn largely from the army engaged in railroad extension work. Courses in irrigation engineering were introduced in the engineering schools shortly after. Irrigation practice was largely developed at a time when there were no irrigation codes, when the standard of water measurement was the variable miner's inch, and before the measurement of the streams started. This measurement was begun as a result of the over-appropriation of natural stream flow.

EARLY STRUCTURES INADEQUATE

Considering the handicap of lack of funds, it is not surprising that the capacity of early structures was commonly inadequate. This inadequacy was also partly due to the attempt to conform to state laws as these were enacted. Imbued with the desire to stretch the limited water resources as far as possible, the framers of such legislation limited allowable diversions to 1 sec-ft. of water for 70 or 80 acres. Such regulations, if adhered to, would make irrigation farming impracticable except on highly retentive soils or by the elimination of losses from canals and laterals. Fortunately, numerous legal means were found, in court actions regarding water rights and in stream administration, to correct this situation.

Although irrigation development in the United States as a whole is probably not far from half completed, the remaining construction required for the irrigation of new lands and for the improvement of the water supply in areas now inadequately supplied, involves numerous long canals of large capacity. Initial provision of excessive capacity in these canals would impose a severe, if not insurmountable, financial burden; while subsequent enlargement would be extremely costly. Therefore the problem of proper canal capacity is a fundamental one.

TWO-THIRDS OF DIVERTED WATER LOST

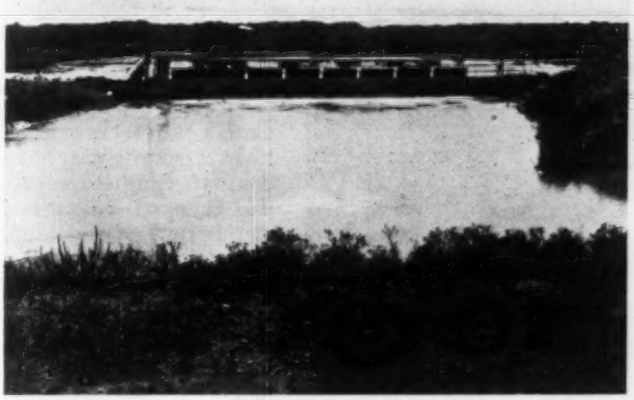
The function of any irrigation system is to provide the moisture required to grow and mature crops. Provided there is sufficient water at the source, the success of the system is measured by its ability to transport water at the time it is needed for the optimum crop. The bare needs of the plant for its growth can be determined within moderate limits by experiment. Considering such plant use, the average irrigation system is only 35 per cent efficient, the rest of the diverted water being lost or wasted in canal operation, by percolation and evaporation from canals and laterals, by seepage to groundwater due to excessive application by the irrigator, through waste by the irrigator past his farm, and by evaporation from the soil.

By far the largest loss, which averages roughly one-half the total, is that of percolation from canals and laterals. Where water is of sufficient value to warrant the additional expenditure required, this loss is reduced to negligible amounts by the use of impervious linings or piping. Of about equal importance are operating losses, soil evaporation, and losses from percolation to the water table. All these are to a large degree controllable without the necessity for addi-

NEEDLE VALVE IN OPERATION
Bully Creek Siphon, Vale Project, Oregon

tional investment. By rigid adherence to a system of rotation in deliveries, operating waste by the management can be reduced to negligible amounts. By strict attention to his irrigation, the farmer need waste but little on his farm. Both losses may often be materially reduced by carefully planned interconnecting drains and ditches to make re-diversion possible where topography permits. Loss to groundwater can be minimized by the use of the largest practicable irrigating heads and suitable runs, so that satisfactory replenishment of soil moisture will be effected throughout the run before the water sinks below the depth of root penetration at any point. Flooding is superior to furrow irrigation in that it permits the water to travel more rapidly, but it is not suited to all crops and soils and it increases soil evaporation.

Evaporation from the soil can be reduced by dust mulching when the character of the crop permits and by reducing the number and length of irrigation periods.



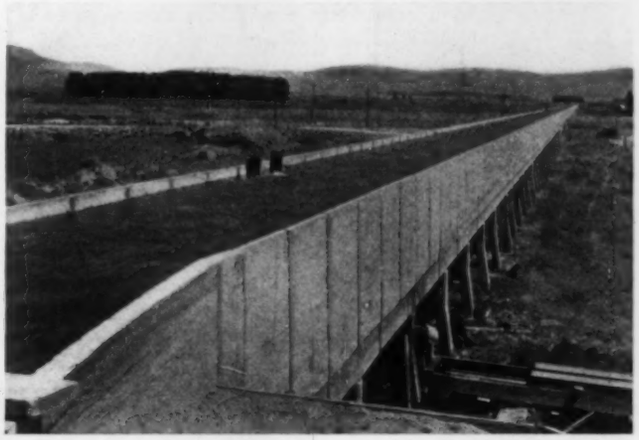
HEAD-GATES OF THE TRI-STATE CANAL
Near Henry, Nebraska

Evaporation from canals and laterals is small compared to other losses but it is often greatly augumented by the growth of useless weeds, brush, and trees along the canal banks.

NORTHERN PROJECTS REQUIRE GREATEST CANAL CAPACITIES

Determination of the proper canal capacity for a project to irrigate new land requires carefully balanced judgment in agricultural, engineering, and economic matters. With an ample water supply available to the irrigator whenever wanted, in the desired quantity, and at a cost that does not materially influence operations, momentary demand for delivery to the individual farm requires consideration of soils, of the crops to be grown, and of irrigation practices. It is not uncommon to find a locality suited almost exclusively to the growing of alfalfa. The first cutting, especially, is likely to mature at the same time throughout the project, so that a uniform practice will develop of irrigating at a certain time with reference to the harvesting of the crop. The result will be a demand throughout the project for a large amount of water within the space of eight or ten days, followed by practically no demand for possibly two or three weeks. The other extreme is found on a sub-tropical project where crops are highly diversified so that planting and harvesting are almost continuous and only a part of the project is being irrigated at any one time.

Although it seems incongruous, the result is that canal capacities on northern projects, where the growing season is short, must be relatively much greater than on southern projects with all-year irrigation. If the cost of water is not a controlling factor, the irrigator applies it so as to produce crops at the lowest cost, often without heeding even the danger signs of rising groundwater and decreasing fertility due to excessive use. At times,



FLUME IN "C" CANAL
Klamath Project, Oregon and California

much as he may want to, the farmer noting such signs cannot afford to level his farm, improve the water-holding capacity of the soil, shorten the irrigation runs, or line his laterals to avert his eventual elimination by rising groundwater. Drainage may come to his rescue, if he can afford it, but when his condition is so relieved he usually increases the use of water still further.

DATA ON CANAL LOSSES INSUFFICIENT

When a reasonable delivery to the individual farm has been decided upon, it becomes necessary to make allowance for canal losses. The physical character of the materials through which the canal is to be built can be compared with those on constructed canals in other places, but vital differences in other factors may enter into the situation, such as silt-laden water. A little silt will seal the banks against loss, but much will reduce the carrying capacity of the water between cleaning operations. Or the water table may rise to cut off the escape of water from the ditch. Many data have been accumulated on losses from selected sections of canals in operation, but the description of the materials through which these ditches pass is far from exact.

When the value of water is a factor, the consideration of methods for preventing its waste becomes immensely more important. The irrigator must invest more, both in preparing his fields and ditches and in applying his water. The canal system may be lined, although, with clear water, lining often introduces difficulties due to aquatic growths attaching themselves to the sides of the canal, thus calling for increased area of section to offset the impaired velocity of the water, and for increased maintenance and operating costs.

The assumption of an ample supply of water at all times is seldom realized. Consideration must then be given to the delivery of a large volume in times of plenty for storage above ground, within the root zone, or deep in the ground for subsequent recovery in time of need. The problem of canal capacity then becomes complex, requiring comparative cost estimates and financial studies. Where the project is to provide an additional supply in

an area already under irrigation in whole or in part, the problem of canal capacity is greatly simplified, at least as regards crop requirements and farm practice, but the other problems still remain.

CONSIDER THE OPERATOR

A large part of our irrigation systems are operated by men without college training in engineering. As a class they are inarticulate, and it is not often that we hear of the woes of the operator. Too often irrigation works have been planned and constructed by engineers without operating experience and without an instinctive understanding of the problems met in operation, human and otherwise. The result is always impaired capacity.

BULLY CREEK SIPHON AND WASTEWAY
Vale Project, Oregon

Faulty alignment of earth canals will bring about erosion of the banks. The necessary protection costs far more than the initial saving in construction cost. In lined canals, where the velocity is high, the same error causes disturbances in the flow, preventing conveyance of the quantity of water intended and introducing serious difficulties when water slops over the sides. Faulty design of transitions, bridges, and other sections, causes loss of gradient and therefore of capacity.

Inclusion of lands that are too high makes necessary the construction of checks. These impair the gradient of the canals, which were designed without taking them into consideration. Improper location of cross drainage and inadequate provision for it often result in frequent clogging of canals by detritus brought in by storms, the removal of which entails a continual and burdensome expense. This impairment of capacity usually occurs at the most inopportune times.

Canals are usually constructed with a sectional area theoretically capable of providing exactly the designed capacity and, in the case of canals in earth, a non-scouring velocity. Growing vegetation on the banks catches the finer materials in the water and brings about encroachment, while the heavier materials transported lodge in the bed. Unless an allowance has been made in the area of the cross section for impaired capacity, frequent cleanings at high unit costs are necessary. Such an allowance, initially provided, would pay handsome dividends by permitting greater accumulation and less frequent removal, at lowered unit costs. If this allowance is adequate, the necessity of exposing virgin soil at each cleaning is eliminated, as well as the attendant increases in percolation losses, which occur until the surface is again sealed by deposited silt. If the sections are lined and the velocity is higher, the difficulty arising from silt deposits is usually absent, but a dffferent problem is presented by the rapid growth of mosses, where the water is clear enough.

Interesting, and at times novel methods have been devised to combat conditions productive of impaired capacity. One such might be briefly described. The main canal of the West Extension Division of the Umatilla Federal Project in north-central Oregon passes through a sandy section subject to heavy winds. Although this canal was lined with concrete, sufficient velocity could not be provided to carry the heavy sands to the wasteways at suitable sites, nor were such velocities desirable, as too much sand would then be carried to the farms. Eventually the operators devised a templet of the size of the canal, which was suspended across it so as to allow a small clearance all around. When the templet is in this position, the escaping water has sufficient velocity to dislodge even the heaviest sand immediately ahead of it. The templet is then moved forward gradually, causing the dislodged sand also to move forward until a wasteway is reached, where the sand can be sluiced out.

The Committee on the Duty of Water in Terms of Canal Capacity plans to make an extensive inquiry by questionnaire to obtain basic data on existing projects. The arrangement of these data in a form useful as a guide in future work has not yet been given sufficient consideration to permit a statement of it. The art of irrigation engineering has not reached a stage where it is reducible to exact mathematical formula, nor does it appear likely that it soon will.

The committee takes this opportunity to invite the members of the Society to submit accounts of interesting experiences that may be helpful in the preparation of its forthcoming report, intended to cover both the determination of proper canal capacity and methods of maintaining this capacity in systems under operation.

Modern Short-Span Bridges

Recent Progress in the Use of Timber, Steel Piling, and Reinforced Concrete

IN THE past decade, highway bridges have been given more detailed attention as the density and weight of automobile traffic have increased. Western engineers have adapted several European methods of concrete arch design and construction—the Freyssinet system of rib-stress compensation, the use of temporary construction hinges, and the tying of arches through the floor system to eliminate abutment thrust. The abundance and relative cheapness of timber in the Northwest have resulted in a return, in the state of Oregon, to the use of pile-bent viaducts built of pre-framed and treated timber, with reinforced concrete decks and composite hand rails. These developments are covered in the paper by Mr. McCullough.

In Nebraska, the absence of bedrock at reasonable depths has compelled the development of a special type of deep foundation capable of resisting the terrific scour that takes place during high water. As explained by Messrs. Mason and Ogle, the problem has been solved by using structural steel shapes as piles to support multiple spans, and by giving the bridges shorter channel lengths and longer approach fills.

The aggregate cost of the short-span bridges erected in this country probably greatly exceeds that of the more monumental structures. In these times of financial stress, new economies have been effected in short-span concrete bridges by utilizing the principles of continuity and restraint. As pointed out by Mr. Beretta, in the third article of this symposium, too great standardization has shut the eyes of engineers to their opportunities for developing and improving the design of short-span structures so that each member will do its full share of the work. Specific examples are given of continuous-span and restrained-beam construction in Texas.

The original papers from which these three articles have been prepared were read by their authors before the joint session of the Structural and Highway Divisions of the Society on July 7, 1932, at the Convention in Yellowstone National Park.

Western Practice Utilizes New Types

By CONDE B. MCCULLOUGH

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DURING the past few years, considerable advance has been made in structural practice throughout the entire country. In the West some of the new developments have been imported from Europe and the eastern United States, and others are directly of Western origin. Among the former may be mentioned the Freyssinet system of arch-rib compensation and adjustment, employed for the first time in the United States at the Rogue River crossing in Oregon; the use of temporary construction hinges in reinforced concrete arches, as exemplified in a number of structures built on the Pacific Coast during the past few years; and the development of the "tied" or "bow-string" arch, used in several structures over coastal streams in Oregon.

All these arch types were originated in Europe or the eastern United States; in Western practice they are importations and adaptations rather than original developments. Before passing to a discussion of methods actually developed in the West, it may be of general interest to consider briefly the imported systems and to enumerate their peculiar advantages.

The first bridge in the United States to be constructed in accordance with the Freyssinet system was that erected across the Rogue River on the Oregon Coast Highway. The theory underlying Freyssinet's method of arch compensation and adjustment and the construction methods employed have been frequently described in American engineering literature during the past few years. In brief, the system consists in the severance of the arch rib at the crown section and the introduction at this point of a system of arbitrary stresses or strains whereby the rib may be placed in its most advantageous elastic state.

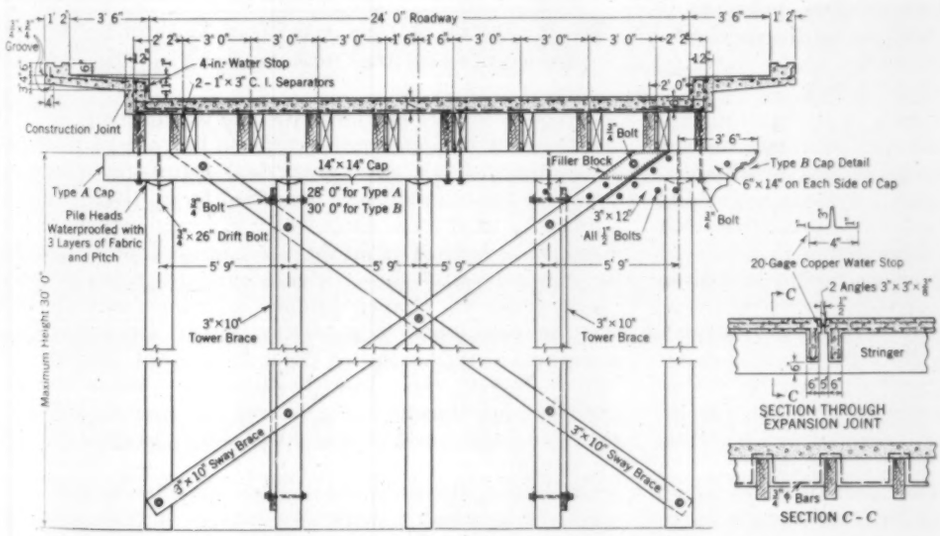

24' 0" Roadway
4-in. Water Stop
2 - 1" × 3" C. I. Separators
Construction Joint
14" × 14" Cap
Filler Block
Type B Cap Detail
6" × 14" on Each Side of Cap
Type A Cap
Pile Heads Waterproofed with 3 Layers of Fabric and Pitch
28' 0" for Type A
30' 0" for Type B
3" × 12" All ¾" Bolts
¾" × 26" Drift Bolt
5' 9"
3" × 10" Tower Brace
Maximum Height 30' 0"
3" × 10" Sway Brace
20-Gage Copper Water Stop
2 Angles 3" × 3" × ⅜"
Stringer
SECTION THROUGH EXPANSION JOINT
¾" ϕ Bars
SECTION C - C

FIG. 1. DETAILS OF TIMBER TRESTLE WITH CONCRETE DECK

The benefits inherent in

this method are briefly as follows: elimination of various stresses—those due to elastic rib shortening under dead load, to early temperature changes resulting from hydration, to early shrinkage, to any initial plastic displacement of the supports, and to early plastic deformation of the concrete proper; ability to counteract the effects of future shrinkage and plastic flow; ease in decentering; and the practicability of releasing centers for re-use at earlier dates than would otherwise be possible. Construction experience on the Rogue River crossing gave evidence of the remarkable ease with which an arch rib may be manipulated by means of hydraulic crown jacks. It is a safe prediction that this system will find increasing favor with American engineers, especially for the longer spans and lower rise-span ratios.

BRIDGE ACROSS THE ROGUE RIVER, OREGON COAST HIGHWAY
First Freyssinet Arch to Be Built in the United States

Temporary construction hinges of either the Considere or Mesnager type accomplish in part the same purpose as Freyssinet's jacks. The Considere or Mesnager method converts the span into a three-hinged arch under all load conditions prior to the period of final fixation, thus eliminating stresses due to temperature changes from early hydration, effects of early shrinkage, initial plastic displacements of supports, elastic shortening of ribs due to all loads placed prior to fixation, and early plastic deformation in the ribs proper. Unlike the Freyssinet system, the Considere or Mesnager method provides no means of compensating for future stresses, nor does it have the definite determinateness of the former method.

BOW-STRING ARCH IN USE

The "tied" or "bow-string" arch type has found a useful application in coastal stream crossings on the Pacific Coast, where the high maintenance cost of painting puts steel truss construction at a disadvantage, and where foundation conditions make it imperative that pier reactions be determinate and vertical.

A structure of the "bow-string" type, built over Wilson River in Tillamook County, Oregon, is illustrated. The cost of such a bridge is only slightly in excess of that for a steel truss span of equivalent length (120 ft.); the architectural effect is superior; and, under atmospheric conditions which are conducive to rapid corrosion, the interest on the additional first cost required is more than offset by the decrease in outlay for maintenance.

Probably the outstanding development in bridge design during the past year in Oregon is the increased and more intelligent use of timber for highway structures. The inauguration of an organized effort along this line is probably a direct outgrowth of the economic depression through which we are passing. During periods such as this, public officials are frequently placed under the severest pressure to further the utilization of local materials and the adoption of programs of drastic economy.

The agitation for local materials is doubtless, in large measure, a manifestation of the universal hysteria that accompanies periods such as this one. On the other hand, the introduction of rigid economies is a very salutary procedure. In fact, the intensive effort to get down to earth and study every possible method of stretching the taxpayer's dollar is one of the great ameliorating phases of any depression. It therefore may be said that, notwithstanding the fact that the idea of using timber more extensively in bridge construction was perhaps born of the somewhat selfish interest on the part of a local industry, nevertheless, in so far as it has focused the attention of public bodies in the West on a fuller appreciation of local materials and their more intelligent and economical use, it has served a very beneficial purpose.

During the past year the State Highway Department of Oregon, in cooperation with the engineering departments of the various trade extension bureaus of the lumber industry, has s ent a great deal of time on this problem. At the outset it was conceded by all that if timber were to be used more extensively in bridge construction, it would be only after an intensive study looking toward the elimination of those defects inherent in the conventional types of timber bridges. Among such defects may be mentioned lack of durability; uncertainty in structural detail owing to the ever-present tendency of timber to split, check, and twist out of shape in service; inherent fire hazards; high cost of maintenance for roadway decks; and lack of adaptability to architectural expression.

Obviously, the first two of these defects can be eliminated or lessened in only two ways: by using an approved preservative treatment such as the creosote

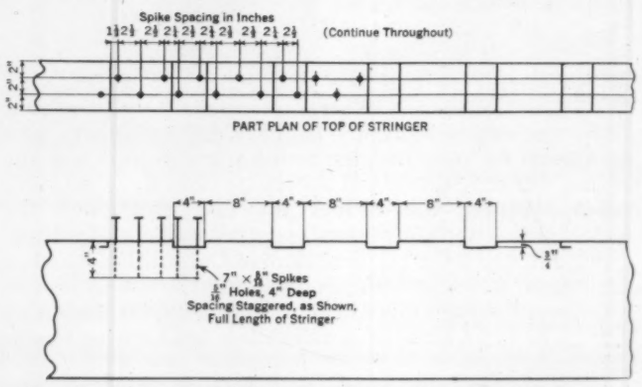
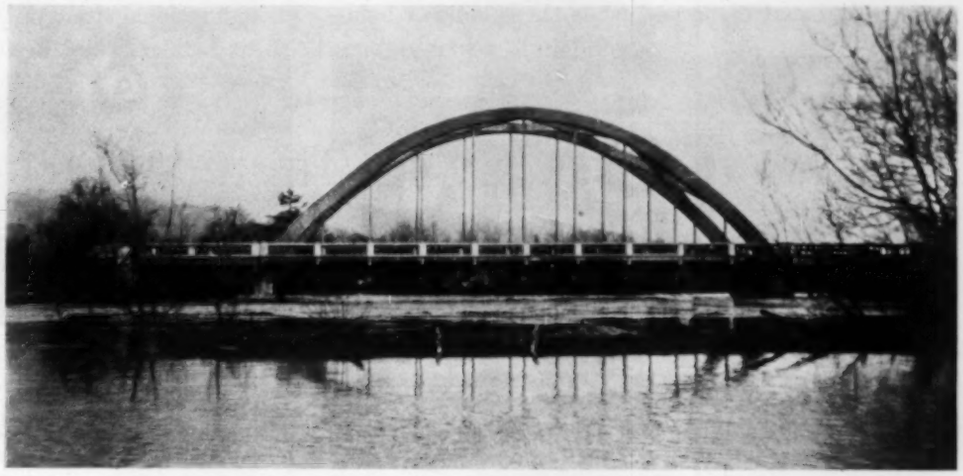


FIG. 2. DETAIL OF DAPS AND SHEAR NAILS IN STRINGERS
Stringers Treated After Holes Are Bored and Daps Cut

pressure process, and by adopting a specification and a system of inspection such as will ensure selection of the highest possible grade of timber for every part of the work. As regards timber preservation, much depends on careful choice of material, its thorough seasoning prior to treatment, and the adoption of a system of incising such as will lessen the tendency towards shrinkage cracks. In order to secure maximum efficiency from treated timber, all the material must be framed and fabricated prior to treatment. Prefabrication and preframing require the preparation of detailed working drawings; in fact, they make it necessary to approach the subject of design of timber bridges from an entirely different viewpoint from that hitherto prevailing.

This new type of timber bridge will not be as cheap in first cost as the ordinary conventional designs but it will be an entirely different structure—one built of selected material carefully seasoned and carefully handled throughout the process of treatment, the utmost attention being paid to methods of reducing checking due to seasoning subsequent to placement. It will be prefabricated and preframed, match-marked, and erected piece by piece in the field in much the same way as steel. Among the various problems that are now scheduled for investigation may be mentioned methods of incising for the prevention of seasoning checks; mixture (oil and creosote) treatments; development of standard details which adapt themselves to prefabrication; and means of careful grading, inspection, and selection of materials.



WILSON RIVER ARCH (120-FT. SPAN) IN TILLAMOOK COUNTY, ORE.
An Example of Bow-String or Tied-Arch Construction

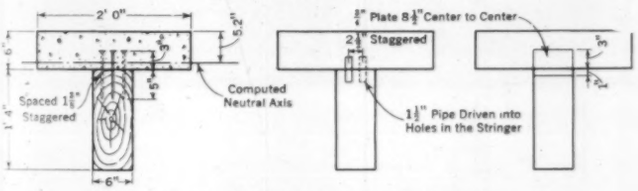


FIG. 3. SOME METHODS OF SHEAR CONNECTION
Under Test at Oregon State College

NEW SPECIFICATIONS DEVELOPED

For short-span construction, the type of timber structure previously described is one which, to a certain extent at least, can be placed in competition with bridges of more permanent materials, such as steel and concrete. Of course, there will always be a field for the ordinary untreated timber structure for temporary construction and at points where alignment and location are not, and cannot, be made permanent at reasonable expense.

With these facts in view, the Oregon State Highway Department—in cooperation with the engineering departments of the trade extension associations representing the lumber industry, national and local—has developed a new set of standard specifications for structural Douglas fir. Among the points in which these specifications differ from those hitherto formulated may be mentioned the segregation of timber into a primary and secondary classification, the primary grade being for the advanced type of timber bridge previously described, and the secondary for use in temporary structures of untreated material. The specifications also provide that all timber must be grade-marked or accompanied by a certificate of inspection from an approved inspection agency.

Thus, through the formulation of a rigid specification, together with careful inspection and an intensive study of approved methods for timber preservation, it is hoped that the inherent defects of timber—its tendency to decay and to split, check, and twist in service—may be overcome, at least to a large degree. The high cost of roadway deck maintenance and, to a certain extent, fire hazard, may be eliminated by combining a reinforced concrete roadway deck with a treated timber substructure and stringer system.

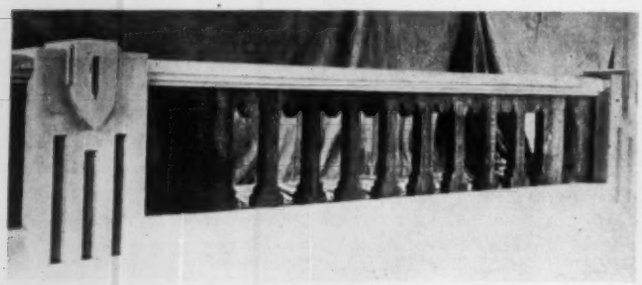
A cross-sectional view of a standard trestle-type bridge developed in Oregon during the past year is shown in Fig. 1. Although the use of creosoted timber in combination with a concrete roadway deck is not new, this illustration exemplifies a few details which are perhaps worthy of note. Among these may be mentioned the placement of the tower bracing on the inner piles of each bent, in order to improve the general appearance of the structure; and the employment of concrete bridging at mid-span.

An alternate method of treating the ends of caps on trestles, where the location warrants some effort for the sake of improved appearance, is also shown. In this arrangement, the sway braces are dropped back to a point underneath the roadway deck and a 6 by 14-in. member, bolted to each side of the cap, is extended from the sway-brace intersection to the end of the cap and finished in curved form. Underneath the 6 by 14-in. members, two 3 by 12-in. planks similarly treated are attached, one to each side of the pile. This detail serves a utilitarian, as well as an architectural, purpose in that it forms a sort of shed over the pile head, which has previously been waterproofed with three layers of fabric and pitch, as shown in the detail.

TIMBER SOLVES SHORT-SPAN PROBLEMS

This type of trestle is designed for long service and is comparatively expensive. Therefore it was felt that special consideration should be given to the treatment of the hand rail, which in most cases is the only part of the

structure visible to the public. A photograph shows a design for a composite rail utilizing reinforced concrete posts and rail cap with balustrade members of timber, sand-blasted and stained brown. This type presents distinct advantages and is economical. The cost of casting the concrete posts and rail cap is not materially greater than that of timber, while the spindle or balus-



COMPOSITE HAND RAIL FOR CONCRETE-DECKED TIMBER TRESTLES
Concrete Posts and Rail Caps with Timber Balustrade

trade members are much less expensive in wood than they would be in concrete. Thus the two materials are utilized where each can be employed to greatest advantage.

There are some sites where the expense of a concrete roadway deck is not warranted and yet the location demands that some attention be paid to architectural detail. For such conditions an all-timber railing may be used. A type recently constructed in eastern Oregon is illustrated.

Thus, by careful grading and selection of structural timber; by adoption of approved methods of preframing, prefabrication, and preservative treatment; by the use of timber and concrete in composite construction; and by the design of railings and balustrades somewhat more ornamental than those heretofore used, it has been possible to develop a short-span highway bridge which eliminates many if not all the defects inherent in the ordinary timber trestles of the past, and which, as regards first cost and maintenance, bids fair to compare favorably with concrete-trestle or viaduct construction.

CRACKING OF FLOOR SLABS STUDIED

One of the defects inherent in the composite type of construction, and one that the Oregon Bridge Department is seeking to overcome, is the cracking of the roadway slab due to stringer deflection. In Fig. 2 is shown a detail employed in this type of construction, by which it is thought to establish a shear connection between the creosoted timber stringers and the concrete roadway deck so as to convert the entire superstructure into a semi-monolithic whole by making the concrete deck act as a T-beam flange for the timber stringer stem. This particular detail employs a system of daps in connection with 7 by $^{15}/_{16}$-in. spikes.

This department, in cooperation with the Oregon State College, has inaugurated a series of tests of full-sized beams (15-ft. span) employing other types of shear connections. In these the shear connection is established by various combinations of daps, spikes, pipes, and plates. The pipes, $1^1/_2$ in. in diameter and 3 in. long, are driven into holes in the top of the stringers; and the plates, which are $^3/_4$ in. thick, are driven into notches across the tops of the stringers. In Fig. 3 are indicated a few of the types that will be tested.

In addition to bending tests, it is planned to include in this research program a freezing and thawing test on full-sized beams in special refrigeration rooms for the purpose of determining the effect of changes in temperature on the shear connection. In view of the large variation in the thermal coefficient for the two materials, it is felt that the utility of this detail will depend in a large degree on the possibility of developing a connection to withstand temperature changes and at the same time possess enough residual strength to transfer the horizontal shear

AN ALL-TIMBER HAND RAIL SHOWING ARCHITECTURAL DETAIL
Inexpensive Construction with Pleasing Appearance

in bending. It will be noted from Fig. 3 that the computed neutral axis for the test beams lies very close to the junction of stem and flange, so that the shear at this point will be nearly the maximum.

SHEAR CONNECTIONS DECREASE DEFLECTION

Because of uncertainty as to the strength of this connection, it is undesirable that any great dependence be placed on it in designing. It is doubtless prudent, for the present at least, to proportion the treated timber stem in such a way that it will carry the entire load as a rectangular beam.

The principal utility of the shear connection is the marked decrease in deflection achieved, which prolongs

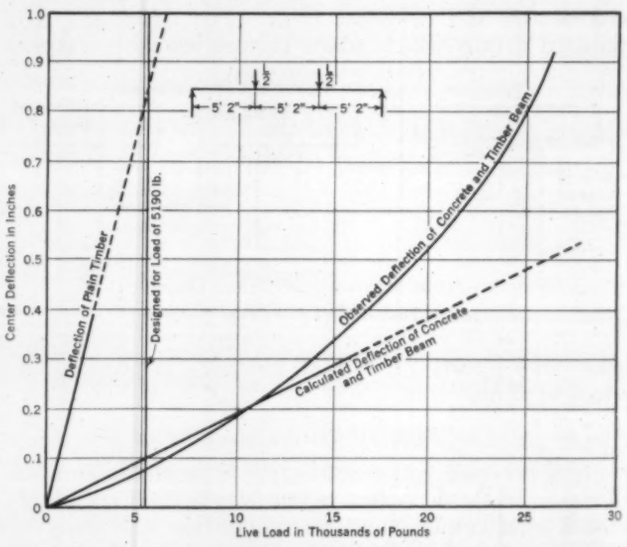


FIG. 4. RESULTS OF DEFLECTION TESTS ON A PLAIN AND COMPOSITE TIMBER BEAM
From Bulletin No. 37, University of Washington Engineering Experiment Station

the life of the roadway deck. That such a decrease in deflection may be expected is evidenced by tests made some time ago at the University of Washington by George Danforth Burr, Assoc. M. Am. Soc. C.E., and published by the Engineering Experiment Station of the University of Washington as Bulletin No. 37. The shear connection employed was a system of $^3/_8$ by 8-in. spikes. The diagram, Fig. 4, which is adapted from the bulletin, indi-

cates the observed and the calculated deflection of the composite beam and the deflection of a plain timber beam of the same dimensions.

TIMBER AN EFFECTIVE BRIDGE MATERIAL

The subject matter of this paper has dealt in a general way with several important advances in the design of highway bridges, but the bulk of it has related to developments in the design of timber bridges for the reason that these are quite recent and have not been covered in any previously published engineering report. While these developments are as yet largely in the experimental stage, results thus far indicate that, through such methods and expedients as have been described, the short-span timber highway bridge of the future will find a growing sphere of usefulness.

Steel-Pile Foundations in Nebraska

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NEBRASKA is traversed by many rivers, most of them wide and shallow. It contains 19 named rivers, which have a combined length of 3,300 miles within the state and are crossed by 261 bridges. Over the Platte River, including its north and south branches, there are 62 bridge crossings, an average of one every 9.4 miles. Yet none of these bridges is founded on bedrock.

A discussion of bridge construction in Nebraska can best be introduced by a brief review of its geological history. The glaciers made at least two distinct advances in Nebraska. The first obliterated all the then-existing drainage ways and filled the valleys with subglacial gravel, making a layer of drift about 100 ft. thick, which is called the Nebraskan deposit. The retreat of the ice was succeeded by a warm period, during which a sheet of silty clay was spread over this drift.

FOUNDATION PILES FOR GOTHENBURG BRIDGE ABUTMENT DRIVEN BELOW THE SCOUR LINE

Pile Tops Were Burned Off Before Crimping Became Severe and Driving Was Then Continued

Then came a second ice invasion, which caused another deep layer of sand and gravel to be deposited to the west of the ice and also laid down a thick deposit of drift under the ice, called the Kansan deposit. After this second ice sheet melted back, there followed a long period of erosion, which resulted in opening up the present valleys. As time went on, there were periods of drought, when thick layers of loess were blown over the deposits of drift and the sand and gravel to the west of them. Later, when the amount of rainfall increased, the present soil of the central and eastern counties was built up over the loess and drift and on top of the alluvial deposits in the valley bottoms.

The Platte River is wide but shallow; its natural banks are generally from 2,000 to 5,000 ft. apart in a valley that varies in width between 3 and 16 miles. It has a drainage area at Columbus of about 56,000 square miles. The middle section of its course in Nebraska goes completely dry in mid-summer for a distance of 170 miles, and yet it has a maximum recorded daily discharge of some 51,000 sec-ft. The average high stage is about 5 ft. above low water, or the river bed.

In the early days, bridges over the Platte consisted of crossings from bank to bank, of timber trestle and floor construction. Only one-way traffic was considered but occasional turnouts were provided for passing. These early bridges, which were usually from 2,000 to 5,000 ft. in length, were built in the expectation that there would be a certain amount of annual damage due to ice and flood, which would require the reconstruction of from 100 to 500 ft. of bridge per year per crossing.

Owing to the width of the river, fields of ice were produced out of all proportion to its discharge or floating capacity, and when the spring thaws occurred the timber pile structures could not resist the destructive action of the ice.

Observations made during such thaws showed that ice jammed the entire river, holding back the water until it overflowed the banks. Eventually it would open up a channel by sweeping away comparatively short sections of bridge. Flowing channels would then form, in which there would be extremely high velocities, and sand beds would be scoured to depths of 30 and 35 ft. Thus Nature herself demonstrated the method of relief required.

SOLUTION OF THE PROBLEM

Obviously, the problem was to transform the stream at any desired bridge site from a wide, shallow channel to a narrower, deeper section with a higher velocity. In order to accomplish this transformation, two measures were necessary: first, the piers of a bridge over a narrower channel in which deep water of high velocities would be expected and required, would have to be driven down well beyond 30 or 35 ft. in order to secure stability in the deep foundation bed of gravel and sand; and second, the approach fills would require armoring of some type to resist the inevitable wash, both upstream and downstream. According to a rule-of-thumb developed in railroad bridge practice in this section, "For every foot of rise of the river above low water, there is a corresponding scour of from 2 to 4 ft." A typical layout for a Platte River crossing is shown in Fig. 1. Here the method of narrowing the stream and protecting the sand fills is indicated.

The adoption of such a policy, if satisfactory as regards

stability, would have the following beneficial effects: it would reduce the volume of ice; increase the ratio of volume of water to volume of ice; confine the stream and reduce the effect of bank erosion; effect accretion of land; reduce the first cost of the crossing; reduce maintenance cost; and reduce interruptions to continuous year-round traffic. The problem was an invitation to the engineer to fulfill his function of controlling the forces of nature economically.

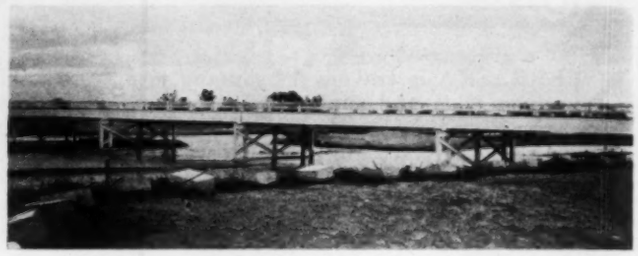
WILLOW MATTRESS PROTECTS APPROACH TO ASHLAND BRIDGE
All Piers Are Steel Piles Encased in Concrete

EARLY BRIDGING OF SMALLER STREAMS

During the period from about 1880 to 1905 each county of the state provided bridges from tax funds, through the agency of a county commission elected for short terms by popular ballot. The custom then was to select good bridge sites and lead the roads to them, and it was not deemed important to skew a bridge. That was the day of light loads, slow traffic, and narrow roadways.

It was considered necessary to provide clear waterways for the smaller rivers and streams; hence there was a preponderance of long-span trusses with short abutments placed well back on the banks. One reason for this partiality for long spans was that deep piers were expensive and difficult to construct, whereas light steel superstructures of long span could be supplied in place at very low cost. Equipment for under-water construction was costly and the hazards were many. The favorite type of pier was the steel tube sunk well below flow line and filled with concrete. Where it was hard to reach a firm footing for such a tube, one to three oak piles were driven through the bottom end of it and the butts were allowed to extend up inside from 8 to 15 ft. The tube was then filled with concrete or crushed stone.

At the present time more consideration is given to alignment and width of roadway, for the requirements as to carrying capacity have been multiplied. Long spans to fulfill present needs would now cost nearly twice as much as the old types. On the other hand, modern bridge-building equipment makes deep-foundation and under-water construction much less costly than formerly. Thus the conventional economic balance between cost of superstructure and cost of substructure now calls for the use of multiple spans instead of long single spans.



TYPICAL STEEL-PILE BENTS, BRACED BUT NOT ENCASED
South Hersey State-Aid Bridge Over the South Platte River

EARLY TYPES OF STEEL-PILE FOUNDATIONS

Examples of early steel-pile construction can still be found in the eastern part of the state. These structures were of the type commonly called a "bedstead" bridge. Most of them were built in the late nineties and therefore are now from 30 to 35 years old. They are of light, pin-connected, Warren type, or Pratt riveted pony-truss design, rigidly fastened to steel columns, or legs, at each corner. The columns are built up with various combinations of channels, I-beams, plates, and lacings, and founded on 12 by 14-in. oak sills laid in deep trenches.

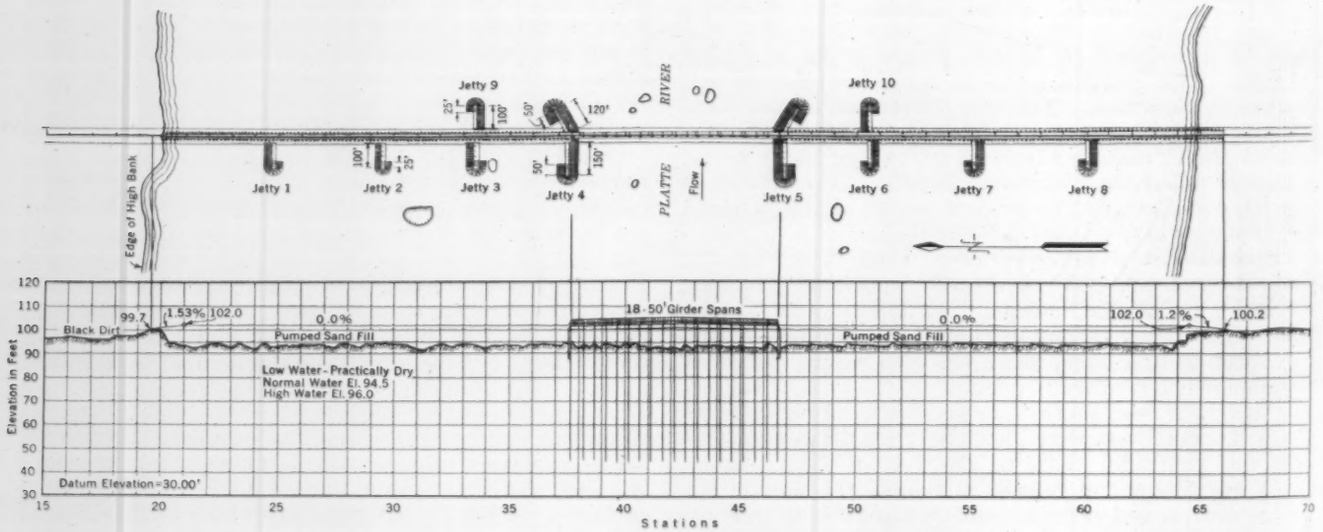
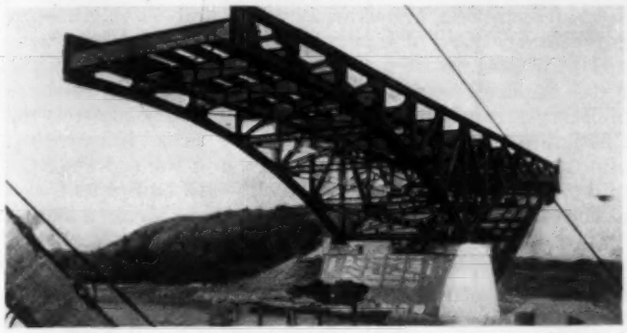


FIG. 1. A TYPICAL PLATTE RIVER CROSSING, ODESSA, NEB.
Channel Narrowed; Approach Fills Armored and Protected with Jetties; Steel Pile Foundations Under Bridge Extended Below Scour Line

In 1901 and 1902 the use of simple I-beam spans became more common. They were supported on 5-in., and then on 8-in. steel I-beams driven as piles and capped with channels and plates. Wing piles were also employed, planks being bolted on to form the backing. In 1908, when wide-flanged sections came on the market, the smaller H-sections were utilized for piling in place of the I-beams. At this time the deep, wide-flanged beams and girder sections were used as through girder bridges, and were divided into panels by floor beams spanned by steel stringers.

Such I-beam spans are still in use for the crossing of very narrow streams, but beginning in about 1919 the girder type just described gave way to the transverse joist girder type for spans between 35 and 60 ft. Simultaneously, H-beam steel pile foundations were developed



VALENTINE ARCH BRIDGE UNDER CONSTRUCTION OVER THE NIOBRARA RIVER

and used extensively by the various boards of county commissioners.

PRESENT STEEL-PILE FOUNDATIONS

From 1912 to 1916 the state provided standard plans for county and state-aid bridges; and from 1919 to date it has not only furnished plans but also taken over the supervision of construction for all state and Federal-aid highway work.

The difficulty of securing deep foundations on the Platte River was overcome by the use of long steel piles. It was found that a section such as the 8-in. H-beam would stand the severe driving necessary to penetrate the deep gravel beds. These piles were driven in single rows as bents, and for protection from corrosion the bents were encased in concrete to a depth of from 2 to 4 ft. below low water. Experience showed that little if any deterioration took place at a depth greater than 18 in. below the stream bed, or groundwater. It also showed that, when the steel was not protected by concrete, the corrosion at an elevation a foot or so above the normal water line was not as great as in the average steel superstructure.

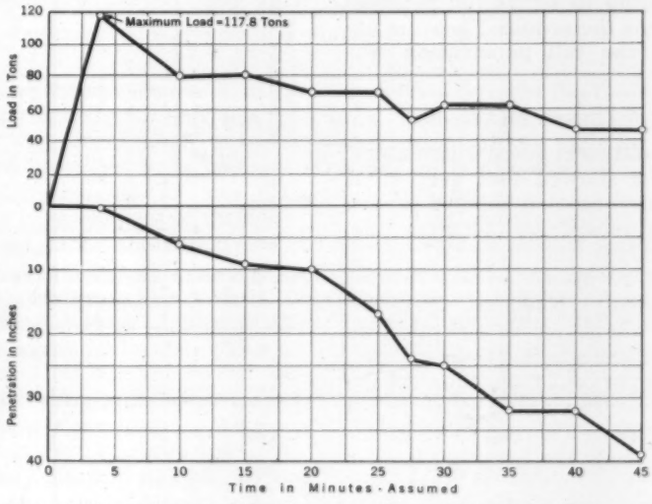


FIG. 2. RESULTS OF PILE PULLING TEST, MACKINAW BRIDGE, ELKHORN RIVER

This 8-In. 32-Lb. H-Column, in 30-Ft. Penetration for 14 Years, Was First Loosened by Driving with a 1,450-Lb. Hammer Having a 15-Ft. Fall. Average Penetration Was $^{7}/_{16}$ In. per Blow

In only very few of the old county bridges having steel pile foundations were the piles encased in concrete or protected in any way except by coats of red lead paint. On a great many such piles examined, it was estimated that the decrease in section had not been more than 1 per cent in 20 years. Exceptions are found in the Salt Creek

CANTILEVER STEEL ARCH OVER THE NIOBRARA RIVER

Central Span of 144 Ft. and Side Spans of 72 Ft. Piers Founded on Forty-One 12-In. H-Piles 50 Ft. Long. Temporary Bridge on the Right Consists of H-Beams Laid Flat

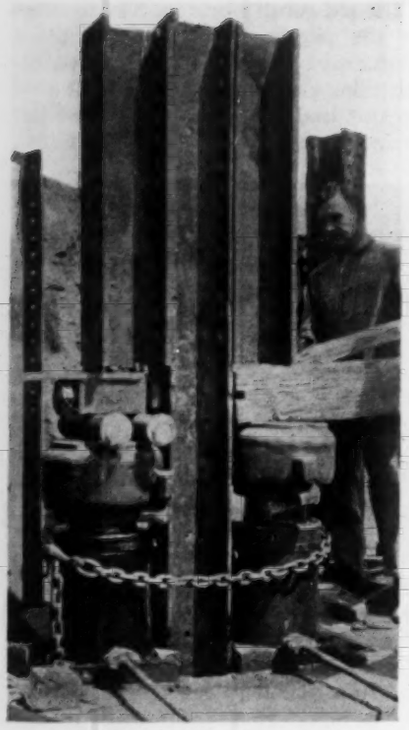
Valley, which is saline to a marked degree. In that locality several old steel bridge legs showed an estimated loss of section of about 2 or $2^{1}/_{2}$ per cent.

MAT PROTECTION ADOPTED

For the protection of the sand approach fills, woven willow mattresses weighted down with rock or concrete were first used. They extended continuously from the river bank out to and around the bridge abutments. Later these mattresses were supplanted by three 6-in. layers of willow placed in alternate directions and tufted every 5 ft. each way with galvanized wire tied to concrete blocks. The top and bottom strands of the galvanized cables, in checkerboard effect, encompassed these mats and were anchored by cables to concrete deadmen in the fill.

This mat protection, parallel to the roadway, did not produce an even scour in the channel abutments, and swift, deep water formed at each abutment. Thus, for a time, longer piles were required in the abutments and in the first or second piers than for the midspans. At the present time, however, a more uniform scour is obtained by placing sand jetties, varying in length from 100 to 200 ft., directly upstream from the abutments. Likewise better protection for the approach fills is now obtained by using similar jetties or retards along them at intervals. These extend upstream and at right angles to the line of the roadway.

To reduce their transverse deflection, the bents of 8-in. H-sections were designed so that alternate piles were turned 90 deg.,



PILE PULLING APPARATUS FOR THE MACKINAW BRIDGE
Results of the Test Are Shown in Fig. 2

thus securing a greater summation of moments of inertia for all the individual piles in a bent. More recently, piers have been strengthened by using 10-in. instead of 8-in. H-sections.

ADVANTAGES OF THE STEEL PILE

The principal advantage of steel piles lies in their ability to stand the punishment of the long-continued hard driving necessary to obtain the required penetration—50 to 65 ft. They have the fortunate characteristic of showing any distress due to driving at the very top. This is probably due to the inertia of the pile and its ability to distribute the impact of the blow at the point of application. Therefore, in a short period of hard, fast driving the pile top will bend and finally wrinkle. This damage usually occurs within the top 6 to 16 in., leaving the remainder of the pile in perfect condition. The damaged end can be cut off with a gas torch in from three to five minutes, and the driving can be resumed with but little loss of time.

By carefully regulating the height of fall of the hammer, the greatest amount of penetration per cut-off can be obtained. The majority of piles require but one cut-off, although there are records of single piles that have been cut off seven times in order to secure full penetration. Obviously, in such cases the required bearing capacity was attained before the full penetration was reached.

DRIVING AND TESTING STEEL PILES

Early attempts to remove old steel piles, where a new bridge was to be constructed, showed that very great effort was required. In many cases a pair of 25-ton mechanical jacks would not loosen an old steel pile. This condition naturally raised the question as to whether strict reliance could be placed on pile driving formulas. Specifications usually consider that the *Engineering News* formula is a criterion of the safe load for a driven pile, and yet the same specifications will permit a test load only 50 to 100 per cent more than the design load. Foundation engineers as a rule estimate the safe load of a pile or, in the case of a spread footing, of a tested area of soil, to be one-half the breaking load or yield point on the load-penetration curve. It is generally conceded, also, that because of point resistance a pile should resist a greater downward load than an upward pull or lift.

In an effort to secure data on this problem, the state inaugurated a policy of investigation which has continued over a period of some five or six years. The loading in these tests was done by hydraulic jacks. After the old piles were pulled the new ones were driven with a hammer and then loaded hydraulically. It was thought that greater accuracy of loading and wider flexibility of load application would be possible by this means than by the conventional platform loading. This method made it possible to test certain piles by alternately loading and pulling, thus establishing the point resistance.

In the Mackinaw or Arlington tests, a load of more than 100 tons was required to pull 8-in. H-sections that had been in the ground 14 years, after they had first been loosened by driving with a drop hammer. The curves of these tests in Fig. 2 show the tenacity of such piles in resisting removal.

The tests at the Gothenburg Bridge were made on 8-in. 32-lb. H-section steel piles 50 ft. long, which had been driven to a penetration of 42 and 44 ft. through gravel and clay. These piles could not be forced down with a 170-ton load. Mill scale cracked and peeled during the test. A perfect stress-strain curve plotted from readings of Berry strain gages taken during the test showed the elastic limit to be at about 35,000 lb. per sq. in., which is almost identical with the value given in the report of the mill tests on that shipment of steel. The equipment used in the Gothenburg tests was capable of applying a load of 300 tons.

In the series of tests made at Fairbury, both 8 and 10-in. H-sections were investigated. Five different piles were tested, each one being given from one to five trials. After the piles were driven with a drop hammer, the hydraulic jack equipment was attached and they were forced down from 3 to 7 in. farther.

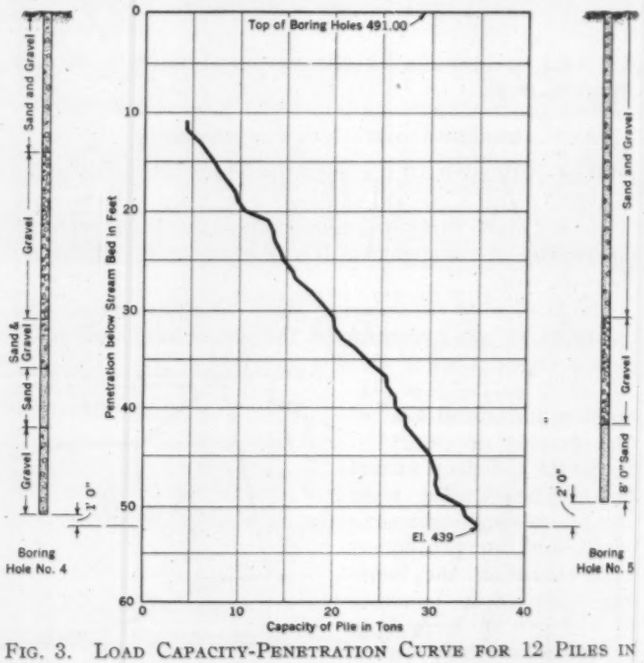


FIG. 3. LOAD CAPACITY-PENETRATION CURVE FOR 12 PILES IN CLARKS BRIDGE, COLUMBUS, NEB.
Average Capacity by Formula for 10-In. H-Section Piles Driven by Steam Hammer

For the two 8-in. H-piles, a safe load or working capacity of about 16 tons each was indicated by the *Engineering News* formula. They required 58 and 81 tons, respectively, to force them down with the jack. The two 10-in. H-sections required 69 and 108 tons, respectively, to move them by the jack, thus indicating a safe working load of 28 tons by the same formula. Another

8-in. H-section pile was first forced down and then lifted up. The last load required to force it down amounted to 63.5 tons, but it was lifted with 40.4 tons. Apparently the point resistance was 23.1 tons. The point was in a clay bed.

The Pender test was made on an 8-in. 32-lb. H-section pile 50 ft. long, which had penetrated 26 ft. so that the point rested in a sand bed. Although its capacity, rated by formula, was only 10 tons, it was loaded with a hydraulic jack to 30 tons, and it held this load continuously for 48 hours without settlement. Then the load was raised to 35 tons without any indication of further settlement. The anchor pile that held half the applied load pulled up, however. Thus the anchor pile withstood a lifting load of 15 tons for 48 hours, but was pulled at a load of 17.5 tons.

The Wood River test was made on an 8-in. 32-lb. H-section 50 ft. long which had penetrated 38 ft., the point resting in a sand bed. By the *Engineering News* formula its capacity was rated as 8.6 tons. It was loaded by the hydraulic jack to 28 tons and held this load for 24 hours without a sign of settlement.

The curve in Fig. 3 shows results of bearing tests made during the driving of piles for Clarks Bridge.

Within the past ten years, 165,700 ft. of 8-in. steel H-section piles have been used by the Nebraska Department of Public Works, and in addition a total of 41,700 ft. of 10-in. H-section piles were used during the year 1931. In the current year, up to April 1932, there were 5,275 ft. of 8-in. H-piles, 12,300 ft. of 10-in. H-piles, and 5,400 ft. of 12-in. H-piles driven under contract. Moreover, there have been hundreds of small bridges with steel pile foundations built in the state through the agency of the various county boards of commissioners. Among all these piles there is no record of a single settlement, washout, or failure.

We wish to express appreciation of the generosity and support of R. L. Cochran, M. Am. Soc. C.E., State Engineer of Nebraska, in permitting the use of drawings and other data, without which this paper could not have been assembled, and also in authorizing the necessary investigations.

Helpful technical advice and assistance in conducting the tests were given by C. M. Duff, M. Am. Soc. C.E., Testing Engineer of the Nebraska Department of Public Works, who also calibrated the hydraulic jacks and furnished other instruments used. Useful historical data, advice, and personal assistance were provided by George E. Johnson, M. Am. Soc. C.E., formerly State Engineer of Nebraska. Data in regard to the geology and geography of Nebraska were supplied by Dean George E. Condra of the University of Nebraska. H. G. Schlitt, Assistant Bridge Engineer in the department, loyally assisted in preparing the illustrations for use with this article.

TWO 150-TON JACKS SET UP FOR MAKING A PILE-LOADING TEST
At Gothenburg Bridge, Platte River

Rigid Frames and Continuous Concrete Spans

By J. W. BERETTA
PRESIDENT, J. W. BERETTA ENGINEERS, INC., SAN ANTONIO, TEX.

IT IS interesting to note that the development of the art of bridge design and construction has not followed a path of steady and continuous progress. Revolutionary additions and changes have come in spurts, between which were long periods that saw little or no new contributions to the advancement of the art.

As new materials became available, the mechanics of their use was gradually reduced to a scientific basis, with consequent improvements in design. The history of bridge design is therefore really a history of the adaptation and use of increasingly better materials. The introduction of wood, stone, iron, steel, and reinforced concrete have each caused periods of notable development. Who can say what the future holds? Perhaps research in the aluminum alloys or concrete of extremely high strength will enable engineers to achieve results never before dreamed of.

Early in their history, bridges began to be grouped into two distinct types. The first includes the magnificent and monumental structures represented in ancient times by the Roman aqueducts and in modern times by such structures as the George Washington Bridge across the Hudson River. The second includes the small, short-span bridges built for strictly utilitarian purposes, which in the past have seldom if ever demanded even passing interest.

Each individual project of the monumental type entails great effort as well as great cost for its design and construction, and consequently has more romantic and popular appeal. However, the aggregate of the many short-span or "workhorse" bridges built far exceeds the investment in the larger structures. Accordingly, and despite the lack of popular romantic appeal, the study and improvement of the design of short-span bridges is an important economic problem, especially in these times of stress.

TYPES AND USES OF SHORT-SPAN BRIDGES

The term "short-span bridge" has a very wide usage and may be applied to an almost indefinite number of types, spans, and combinations of spans. For purposes of clarity and brevity, the term will be used here only for short bridges in which the individual spans do not exceed

MARTINEZ STREET BRIDGE, RIGID-FRAME CONCRETE STRUCTURE, SAN ANTONIO, TEX.
Crosses the San Antonio River with a Span of 101½ Ft.

150 ft. This limit will include most "workhorse" structures.

From the standpoint of design, as differentiated from construction, short-span bridges can be grouped in two specific classes as statically determinate and statically indeterminate. The serious study and adaptation of statically indeterminate design for short-span bridges has been contemporary with improvements in the use of reinforced concrete. Because of the peculiar advantages of this construction material, the application of statically indeterminate design has been almost entirely concerned with multiple-span or rigid-frame continuity.

DESIGN AND ECONOMIC PROBLEMS

The basic problems entering into the design of any bridge, either monumental or short-span, are well defined and of striking similarity. Unfortunately, because of the greater prominence of long-span structures, more attention and effort have been lavished on their design. When careful thought is given to the matter, it is easy to see that the design problems of the smaller bridges are actually as complex and important as those of the larger ones. In the past, engineers have been too prone to forget this and have drifted into standardization, thus excluding the opportunities for research, development, and improvement afforded in long-span design. Small savings in design cost have too often resulted in large wastes of construction funds.

The paramount, though often neglected, problem of bridge design is that of economy. All the material in a structure should be so disposed as to permit maximum efficiency, and each part of a structure should do all the work possible in carrying the imposed loads. Perhaps the next most important aim is the proper adaptation of type to the specific requirements of a particular location. In this connection there are several limiting factors of equal importance: the requirements of loading; composition of the foundation soil encountered; type of crossing; and adoption of the construction methods most familiar to the available labor. For the most part, these are strictly engineering and economic problems, with which the engineer should thoroughly familiarize himself.

Another consideration entering into bridge design is that of esthetic proportion, of beauty, and of harmony with the surroundings. Where true engineering knowledge and accurate mathematical analysis are applied to the problem, the result is practically sure to have a certain symmetry and pleasing proportion. But this is not entirely sufficient; an architectural study should be made to include not only the general outlines of the structure, but also the details, such as railing, bridgehead, span soffits, abutments, and piers. On practically every monumental bridge structure the services of a qualified advisory architect are employed; yet on short-span bridges such a practice is the exception. A comparison of the results obtained gives overwhelming evidence in favor of competent architectural advice.

Short-span bridges must necessarily have maximum headroom consistent with span; maximum span consistent with minimum number of piers; maximum stability against external natural forces; and designed strength, not only for present, but also for future load conditions.

RECENT ADVANCES IN BRIDGE DESIGN

Formerly, structural thought was shackled by the dominant principle that each component part of a structure should maintain a complete separateness of action. When fear or laziness as regards mathematics was conquered and it was found that the design of continuous structures was not prohibitively complicated, there emerged a new concept of structural action, according to which all parts of a structure are considered to act together.

Since practical applications of the principles of structural continuity are comparatively new, very few design data are available in published form. Great credit should be given to Arthur G. Hayden, M. Am. Soc. C.E., Designing Engineer, Westchester County Park Commission, New York, for the evolution of practical design methods and educational promotion of the idea of continuous structures. To date, four practical design methods have been developed, each of which is definitely successful. The first method, as evolved by Mr. Hayden, utilizes analytical calculations which include the use of unit loads and influence lines. The second, also evolved by him, is a mechanical analysis of the structure with the aid of the deformeter of George E. Beggs, M. Am. Soc. C.E. The third method, which I much prefer, is the analytical calculation of the actual stresses in a structure through the use of simultaneous equations. The fourth method, as developed by Hardy Cross, M. Am. Soc. C.E., Professor of Structural Engineering, University of Illinois, is based on the distribution of fixed end

moments by means of a series of approximations founded on stiffness ratios. These methods are all adequately covered in available engineering literature.

Interest in new types of highway bridges is spreading rapidly throughout the United States. In order to illustrate some of the possibilities for progress in this field, several bridges recently designed and constructed in the state of Texas will be briefly described.

THE MARTINEZ STREET BRIDGE, SAN ANTONIO

The San Antonio River, which aimlessly winds through San Antonio from north to south, presents practically ideal conditions for the utilization of rigid-frame bridge design. The channel width, due to the encroachment of years, is comparatively narrow, 70 ft. in the upper reaches and 100 ft. in the lower. The stream traverses not only the most populous sections but also the business center of the city, and is crossed by a large number of streets where grade elevations and widths are definite and unchangeable. Because of its large drainage area, in which the stormwater run-off has been greatly accelerated by roofs, pavements, and other urban construction, it often experiences a rapid and violent increase in flow.

In the case of the Martinez Street Bridge, details of which are shown in Fig. 1, a continuous-frame design was adopted to cope with the problem in hand. The bridge, which spans a 100-ft. channel on a 10-deg. skew, is 101 ft. $6\frac{1}{2}$ in. long from abutment to abutment. It appears from available records that this structure has the longest single span of any reinforced-concrete rigid-frame bridge in the United States. Its over-all width is 54 ft. 9 in., including a 36-ft. roadway and two 8-ft. sidewalks. The total depth of the structure is 38 in. at the center and 65 in. at the face of the abutment.

Full live-load stresses from a standard H-20 loading, plus 30 per cent impact, were used in the design calculations on the assumption that the truck train was carried on any two ribs. Each outer rib carries a cantilevered sidewalk, on which the live load was assumed as 100 lb. per sq. ft.

SCHEME OF DESIGN

In designing this bridge it was early ascertained that the stresses due to dead load were very large compared to those for live load and that the dead load would have to be reduced in every way possible. The concrete mix was designed for a 28-day strength of 3,000 lb. per sq. in., which, following the recommendations of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, allowed a working unit compressive stress in the concrete of 1,200 lb. per sq. in. The reinforcing steel for this design was stressed to 18,000 lb. per sq. in.

An even greater reduction in the dead load was made by the adoption of a cellular ribbed design. In those sections of the rib where the bending moment was positive, the roadway slab was assumed to provide the necessary T-action. Near the abutment, where the moment changed to negative, a soffit slab was added to give the required inverted T-action. To provide a smooth passage for flood waters and accompanying debris, the soffit slab was extended across the span, thus forming the longitudinal cells seen in the photograph.

The abutments were designed with vertical ribs, constructed so as to supply the necessary restraint for the span ribs. To relieve the critical compressive stress at the knee it was necessary to introduce a fillet. This was reconciled with other problems of stress distribution and with appearance by giving the soffit of the bridge an elliptical shape, with a major axis equal to the span and a half minor axis of 2 ft. 6 in. In addition to the fillet, the knee was provided with heavy reinforcement.

The soil on which the structure rests is clay of good quality, carrying a conservative imposed load of 3,000 lb. per sq. ft. Solution of the foundation problem indicated the use of a cantilevered footing with ribs corresponding to those in the abutments. By this means the lack of rigidity in the foundation material was overcome, and the actual weight of the bridge was used to provide that part of the resisting moment not available through horizontal frictional thrust. To ensure sufficient friction, a fin was attached under the base of the foundation at right angles to the footing ribs.

SOME INTERESTING RESULTS

A study of the moment, shear, and reaction diagrams in Figs. 2 and 3 gives a clear idea of the stress distribution through the component parts. Such an intricate

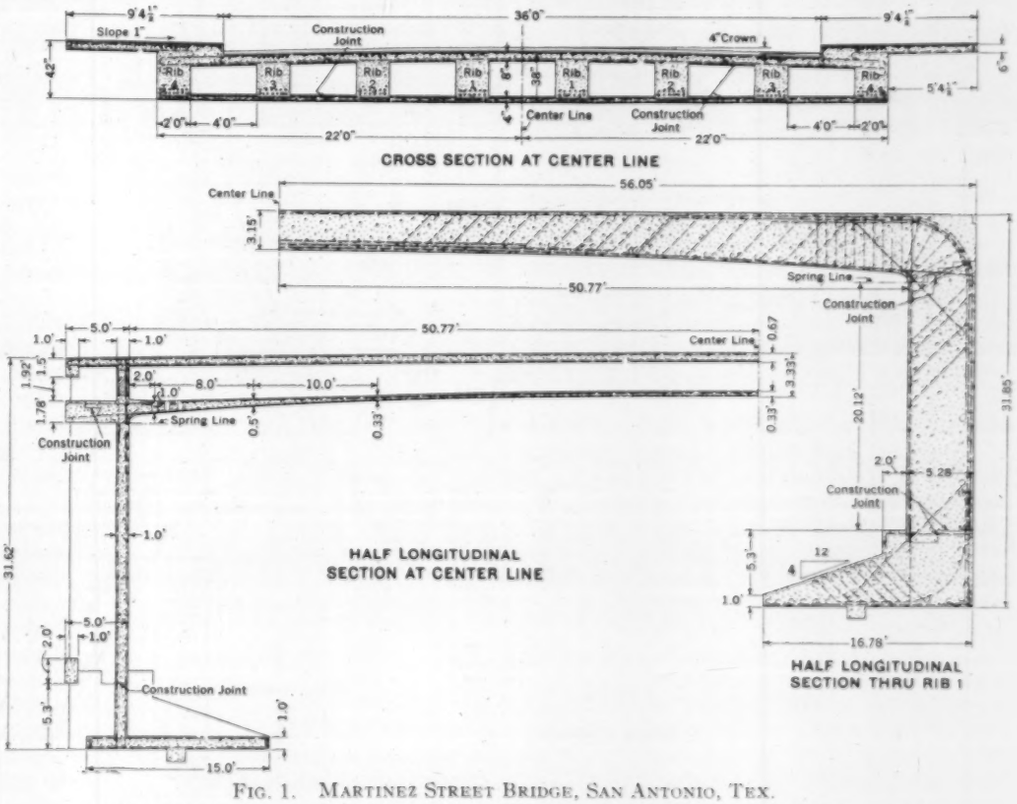
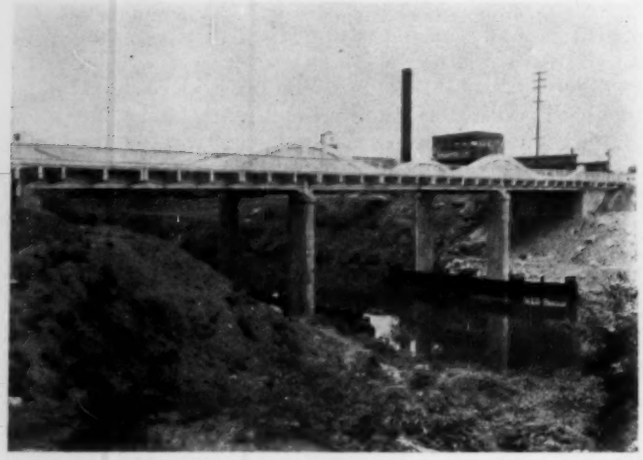


FIG. 1. MARTINEZ STREET BRIDGE, SAN ANTONIO, TEX.
Probably the Longest Single-Span Rigid-Frame Concrete Bridge in This Country

inter-action of all the parts of a structure under varying load conditions provides a striking contrast to older concepts of bridge design. Every part of the bridge does its full share of work in carrying the superimposed loads.



CONTINUOUS CONCRETE BRIDGE CARRIES MCKEE STREET OVER BUFFALO RIVER, HOUSTON, TEX.
Design by J. G. McKenzie, Assoc. M. Am. Soc. C.E.

As a means of achieving proper workability in the mix without excessive wetness, and of gaining additional denseness in the concrete, an admixture was used. Results were successful where the concrete was placed in the forms within 15 minutes, but several times when this time was exceeded the admixture congealed and seriously impeded workability. The concrete for the structure was hauled by truck from a central mixing plant, and some trouble was experienced from segregation until a bin was provided in which the mix was agitated after delivery. Experience gained on this project indicates that agitator trucks should be used for the delivery of all concrete from a central mixing plant, especially when admixtures are used.

The total dead-load deflection after the removal of the forms amounted to only $1^{3}/_{8}$ in., although much more was originally expected. On completion of the structure an initial settlement of $^{1}/_{16}$ in. was noted at the abutments, but no further settlement has occurred. This bridge, now in use for more than a year, has proved to be a thoroughly practical structure. As completed, it cost a total of $38,192.64, including railing and engineering

THREE-SPAN CONTINUOUS BRIDGE, HOUSTON
Almeda Street Structure, Designed by J. G. McKenzie, Assoc. M. Am. Soc. C.E.

contracts. It was designed by the J. W. Beretta Engineers, Inc., under the direction of W. E. Joor, vice-president of the concern.

It was believed that an entirely original course was being followed in the design and construction of the bridge. When the structure was nearly completed, however, documentary evidence was received that proved beyond the shadow of a doubt that there is nothing new under the sun. The February 5, 1931 issue of *Beton und Eisen* carried an article illustrating a bridge having many similar characteristics and striking similarity of detail. It is evident that a similar problem was met and solved in Germany at practically the same time

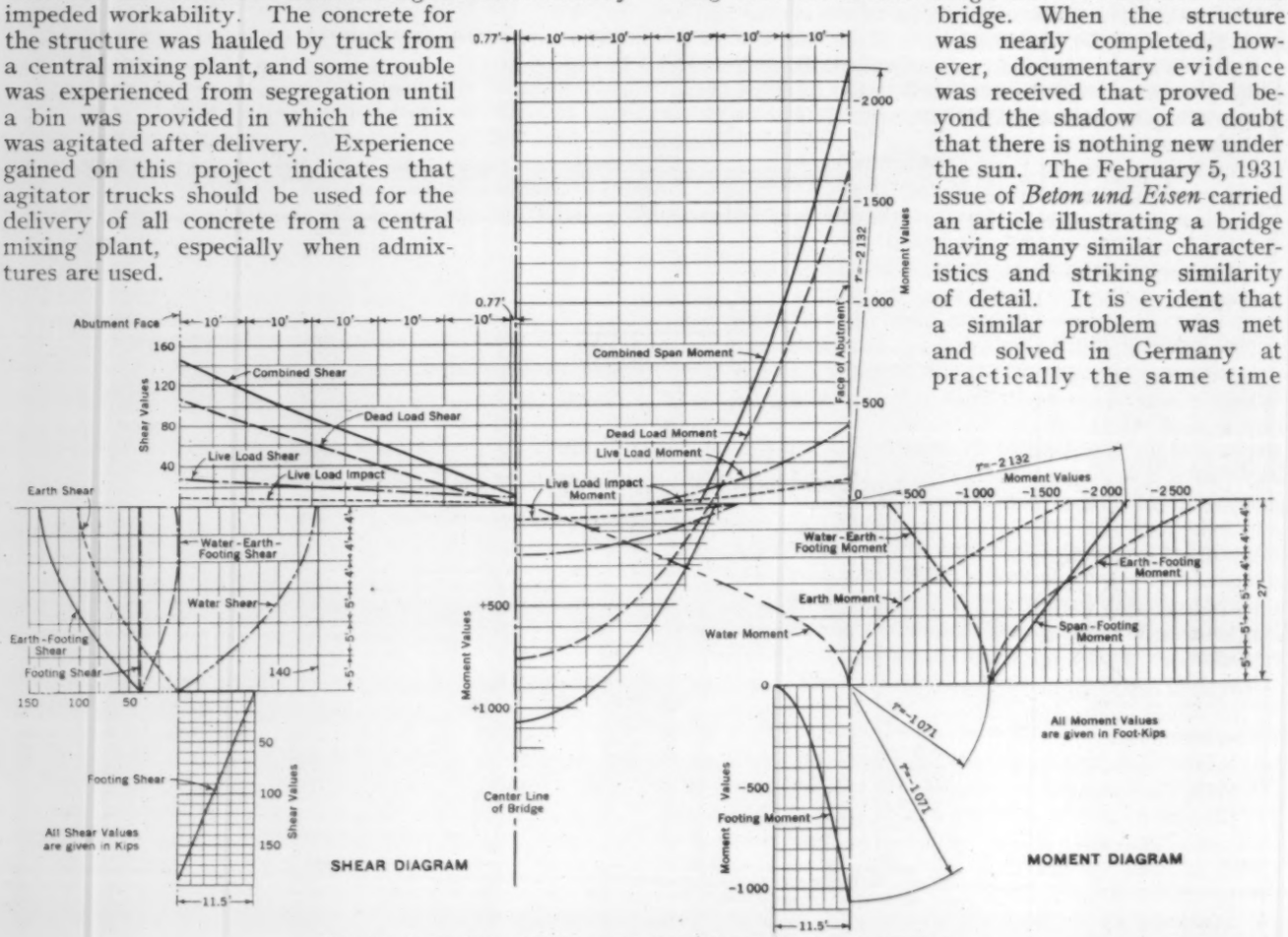
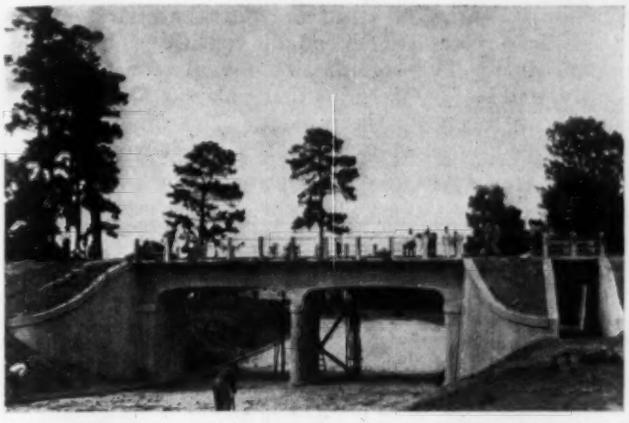


FIG. 2. SHEAR AND MOMENT CURVES, MARTINEZ STREET BRIDGE

that the Martinez Street Bridge was designed, although there was no interchange of ideas between the designers.

The crossing of the Comal River at Garden Street, in the city of New Braunfels, Tex., presented problems considerably different from those at the Martinez Street site. Here the three spans were continuous and rigidly attached to the two central piers. As this structure was



TWO-SPAN RIGID-FRAME RAILROAD GRADE SEPARATION
75th Street Underpass, Houston, Tex.

adequately described in the May 1932 issue of CIVIL ENGINEERING, no further discussion need be given here.

RECENT BRIDGES IN HOUSTON

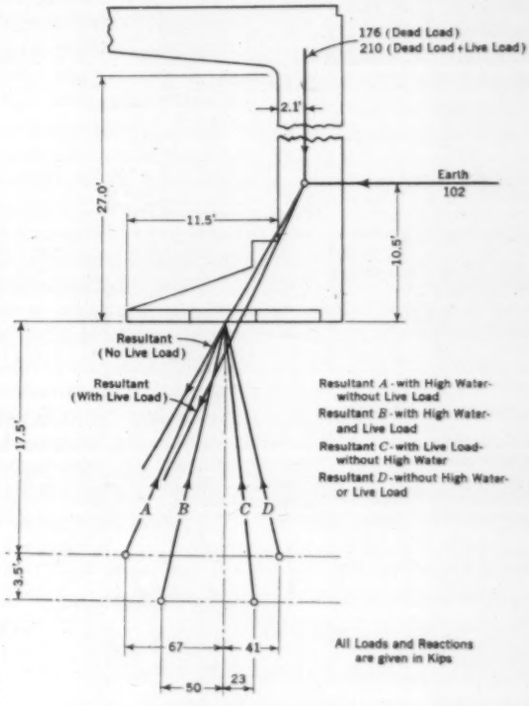
Under the able direction of J. G. McKenzie, Assoc. M. Am. Soc. C.E., the city of Houston has successfully completed four notable structures utilizing the principles of rigid-frame or continuous design. The 75th Street underpass is a two-span bridge of true rigid-frame design. As shown in a photograph and in Fig. 4, it is of the barrel slab type and has two clear spans of 21 ft. and a headroom clearance of 14 ft. above the roadway. It carries a single-track railway supported on ballast and has a walkway about 3 ft. 8 in. wide on each side. This structure was designed for Cooper's E-65 loading plus 100 per cent impact. It is probably carrying the heaviest live load yet placed on a rigid-frame bridge of reinforced concrete construction.

Also in Houston is the McKee Street Bridge, a three-span partial through-girder, reinforced concrete structure with a center span of 120 ft. and two end spans of 85 ft. each (Fig. 5). The center section constitutes probably the longest reinforced concrete girder span in the United States. It is built on a 10-ft. skew with a clear roadway width of 38 ft. 4 in. between girders, and two sidewalks each about 5 ft. wide. Two main girders support the floor beams, which are spaced at about 10 ft. on centers and in turn support the stringers, similarly spaced. The depth profile of the top of the girders follows the bending moment curve, thus giving them an unusual curved appearance. The girders reach their maximum depth over the intermediate supports, where they extend 8 ft. above the roadway slab and 6 ft. below, making their total

FIVE-SPAN CONTINUOUS STRUCTURE UTILIZES HAUNCHED SLAB
75th Street Bridge Over Bayou, Houston, Tex.

depth 14 ft. The design of this structure utilizes horizontal continuity only and includes no rigid pier connections.

A study of the arrangement of the component members of this bridge shows a striking similarity to structural steel design and practice. In fact, many features strongly suggest the translation of a steel design into concrete with decidedly good results. A further similarity to steel practice was the painting of the exposed parts of the girders with aluminum paint, giving a bright, metallic, and highly decorative appearance. The massing of the reinforcing steel in the girders and other interesting details are shown in Fig. 5. Additional details are to be found in the *Engineering News-Record* for July 14, 1932.



REACTION DIAGRAM

FIG. 3. REACTION DIAGRAM FOR THE MARTINEZ STREET BRIDGE
For Various Combinations of Water and Live Load

OTHER CONTINUOUS STRUCTURES

Another structure that is continuous over three spans is the Almeda Street Bridge in Houston, which has a center span of 60 ft. from center to center, and two side spans of 40 ft. each. The roadway is supported on continuous deck girders spaced about 10 ft. on centers, which also support floor beams with the same spacing. The result is that the roadway slab is laid out in square panels. The photograph of this bridge indicates the methods of supporting the girders on the piers.

The fourth bridge is located at a bayou crossing on 75th Street and is also decidedly different from the others in this group. It is of haunched slab type, continuous over five spans, with an additional simple span 22 ft. 6 in. long at one end. The central span is 60 ft. long and is flanked on each side by a 50-ft. span. The two end spans are each 22 ft. 6 in. long. At each intermediate point of support are two slender oblong piers

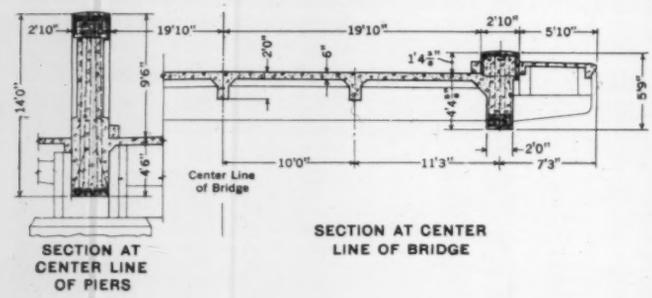
carrying the thickened or haunched slab, which in reality acts as a cross girder. The two outer sets of piers under the continuous spans are designed as rockers, which have a curvature diameter equal to their length. These piers assist in taking care of the expansion of the bridge, which is free to move on both abutments. High above the roadway, which is 38 ft. wide, are two sidewalks each 5 ft. 6 in. wide. These sidewalks, together with the curbs, form a longitudinal conduit for the carrying of pipes and public utilities.

RIGID-FRAME BRIDGE OVER WALLER CREEK HAS SPAN OF 39 FT. Designed by H. R. F. Helland, M. Am. Soc. C.E., for the City of Austin, Tex.

RIGID-FRAME BRIDGES IN AUSTIN

Two rigid-frame bridges over Waller Creek were designed for the city of Austin, Tex., by H. R. F. Helland, M. Am. Soc. C.E., one at 32d Street and one at 34th Street. The 32d Street crossing has a 30-ft. roadway and two sidewalks each 5 ft. 6 in. in width. The bridge is a deck-rib structure of rigid-frame construction, but the soffit has a deep graceful curve so that in appearance the bridge suggests an arch. The clear span is 39 ft. At this particular location the foundation is hard limestone, which presented ideal conditions for a rigid-frame design. By means of a carefully designed substructure detail, a pin-connected condition of the frame at the base is very closely approximated.

The second Austin rigid-frame bridge crosses Waller Creek at 34th Street. In design, limiting conditions, and dimensions it is identical with that at 32d Street. The outside decorative detail, however, was changed to give it a somewhat different appearance.



PRIVATE ENTERPRISE IN BRIDGE DESIGN SHOULD BE ENCOURAGED

Whenever the design and construction of a large, monumental bridge is contemplated, the first step is to secure the services of a dependable consulting engineering organization. This organization is chosen for experience, ability, and integrity, and is then entrusted with the entire responsibility for the project. This practice should be extended to short-span bridges, for it would be the means of fostering originality, economy, and rational development in their design and construction. When private enterprise on a sound basis makes advances in an art, public works organizations should not be slow to take advantage of the progress made.

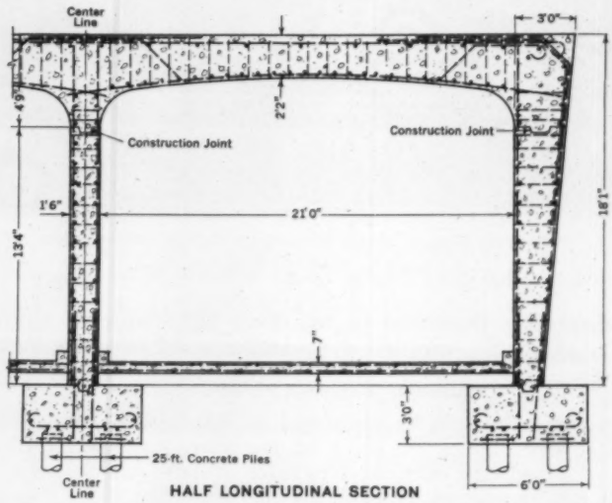


FIG. 4. UNDERPASS ON 75TH STREET, HOUSTON, TEX. Designed by J. G. McKenzie, Assoc. M. Am. Soc. C.E.

Adoption of some policy that would tend to encourage independent research and progress in design would increase the importance and scope of bureaucratic bridge organizations. Instead of producing designs and plans, these organizations might act in an advisory and coordinating capacity on all public works. They would then work in cooperation with private engineering enterprise rather than in competition with it. Through this means public works programs could take full advantage of developments in design, and this progress could be kept practical, consistent, and sound in principle. Such a plan would be in keeping with the policy of the Federal Government of relieving unemployment in the engineering profession through the extension of public works.

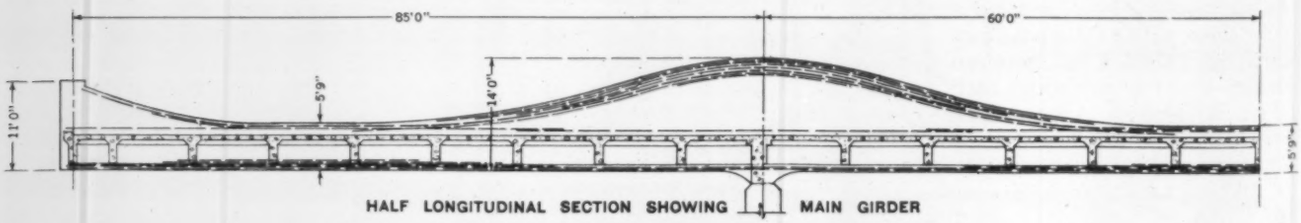


FIG. 5. DETAILS OF THE MCKEE STREET BRIDGE, HOUSTON, TEX. Spans 85, 120, and 85 Ft. Long

Improvement of the Columbia River

A Digest of the Report Recently Submitted to Congress by the Corps of Engineers

By THOMAS M. ROBINS

LIEUTENANT COLONEL, CORPS OF ENGINEERS; DIVISION ENGINEER, PACIFIC DIVISION
SAN FRANCISCO, CALIF.

AT THE Tacoma Convention of the Society, in July 1931, John S. Butler, M. Am. Soc. C.E., gave a preliminary statement of the study of the hydraulic possibilities of the Columbia River, undertaken by the Corps of Engineers in accordance with the provisions of the River and Harbor Act of January 21, 1927. His abstracted paper appeared in the September 1931 issue of CIVIL ENGINEERING. *Now that the Corps of Engineers has finished its study and transmitted its report to Congress, it is possible to publish the conclusions reached and the recommendations made. In the paper by Colonel Robins, presented before the Irrigation Division on July 7 at the Yellowstone National Park Convention and here abstracted, are given the main points of the report, covering the subjects of navigation, flood control, hydro-electric power, and irrigation.*

THE Columbia River rises in British Columbia, through which it flows for a distance of 425 miles. It enters the United States in northeastern Washington, flows southerly to the mouth of the Snake River, and thence westerly between Oregon and Washington, emptying into the Pacific Ocean at a point about 610 miles north of San Francisco Bay and 160 miles south of the Straits of Juan de Fuca. The total length of the river is about 1,200 miles, and its total drainage area about 259,000 square miles. It is tidal as far upstream as Warrendale, about 140 miles from its mouth, or 36 miles above Portland. The river and its principal tributaries are shown in Fig. 1.

In its course to the sea, the Columbia cuts through both the Cascade Mountains and the Coast Range. Between these mountain ranges it traverses an area of lowland consisting partly of the Willamette Valley in Oregon and the Cowlitz Valley in Washington. From the Cascade Range to Rock Island Rapids it has eroded a canyon of varying depth and width in the basalt formation resulting from the great lava flows of eastern Oregon and Washington.

At low stages, the Columbia River falls 1,284 ft. between the Canadian border and tidewater, as shown by Fig. 2. Above the mouth of the Snake the profile is a succession of steep slopes connected by lines of less grade with further concentrations of drop at Kettle Falls, Grand Rapids, Rock Island, and Priest Rapids. The total fall in this section of the river is 975 ft., and the average slope is 2.3 ft. per mile. Between the mouth of the Snake and tidewater the total fall is about 309 ft., and the average slope 1.7 ft. per mile, with concentrations of drop at Celilo Falls and Cascade Rapids.

The mean annual flow of the Columbia at The Dalles during the period 1878–1928 was equivalent to 11.75 in. over the entire drainage area, or about 146,000,000 acre-ft. The lowest year of record produced about 58 per cent of the mean, or 85,500,000 acre-ft., and the highest year showed 154 per cent of the mean, or about 225,000,000 acre-ft. Extremes of flow and mean flow, together with corresponding elevations of natural water surface, for several strategic points along the river, are given in Table I. The maximum record for all time, that of June 1894, shows a flow of 700,000 sec-ft. at the international boundary and of 1,170,000 sec-ft. at The Dalles.

GENERAL DESCRIPTION OF THE WATERSHED

That part of the Columbia watershed within the United States has an area of about 220,000 square miles and embraces parts of seven states: Washington, Oregon, Idaho, Montana, Wyoming, Utah, and Nevada. The bulk of this great area lies between the Continental Divide and the Cascade Range and includes the Bitter Root Mountains. Although many benches and flats exist throughout this basin, the largest extent of open country is the Columbia Plateau, a table land bounded on the west by the Columbia River, on the north by the Columbia and Spokane rivers, on the east by the Bitter Root Mountains, and on the south by the Snake River divide.

On this plateau is located the proposed Columbia Basin Irrigation Project. Its rolling prairies slope in general from northeast to southeast. On the north and east it is indented by many dry stream gorges, of which the Grand Coulee is the most important. This coulee was scoured out through the lava flows by glacial action. It leaves the Columbia and runs through a canyon from one to two miles wide, with walls 400 ft. or more high, for a distance of 33 miles, and then its floor drops abruptly 400 ft. From this point south the coulee is less deep and gradually merges with the plateau.

TABLE I. DISCHARGE OF THE COLUMBIA RIVER, APRIL 1913 TO MARCH 1930

LOCATION	MINIMUM RECORD		MEAN	MAXIMUM RECORD	
	Flow Sec-Ft.	Elevation Ft.	Flow Sec-Ft.	Flow Sec-Ft.	Elevation Ft.
International Boundary	14,000	1,288.4	99,000	450,000	1,333.3
Grand Coulee	17,000	935.0	109,000	462,000	948.0
Rock Island	21,000	548.0	121,000	528,000	589.5
Priest Rapids	21,000	405.0	121,000	528,000	431.0
The Dalles	40,000	43.0	185,000	800,000	102.5
Warrendale	43,000	1.0	190,000	800,000	37.5

It is proposed to use the upper part of the Grand Coulee as a storage reservoir for water pumped from the river to irrigate the Columbia plateau. The bed of the upper coulee is comparatively flat and at an elevation of about 600 ft. above the river at low stage.

DAMS, RESERVOIRS, AND STORAGE

After a large number of possible dam sites on the main river were investigated, ten sites were chosen as best meeting the essential requirements for a comprehensive scheme of improvement. Data pertaining to the proposed developments at these ten sites are given in Table II.

If a low dam is built at The Dalles, all the sites will have to be used eventually. If a high dam is constructed it will not be necessary to make use of the sites at the John Day and Umatilla rapids. Either combination will develop all but about 75 ft. of the total low-water head in the river between the international boundary and tidewater. This loss of 75 ft. in head occurs between Priest Rapids and the mouth of the Snake River. In

this stretch there are no favorable power sites, and raising the water to an elevation greater than 330 at any point below the mouth of the Snake will inundate the towns of Pasco and Kennewick, besides causing extensive damage to other property. This has been an important factor in the decision to omit from present consideration development of the fall in this section of the river.

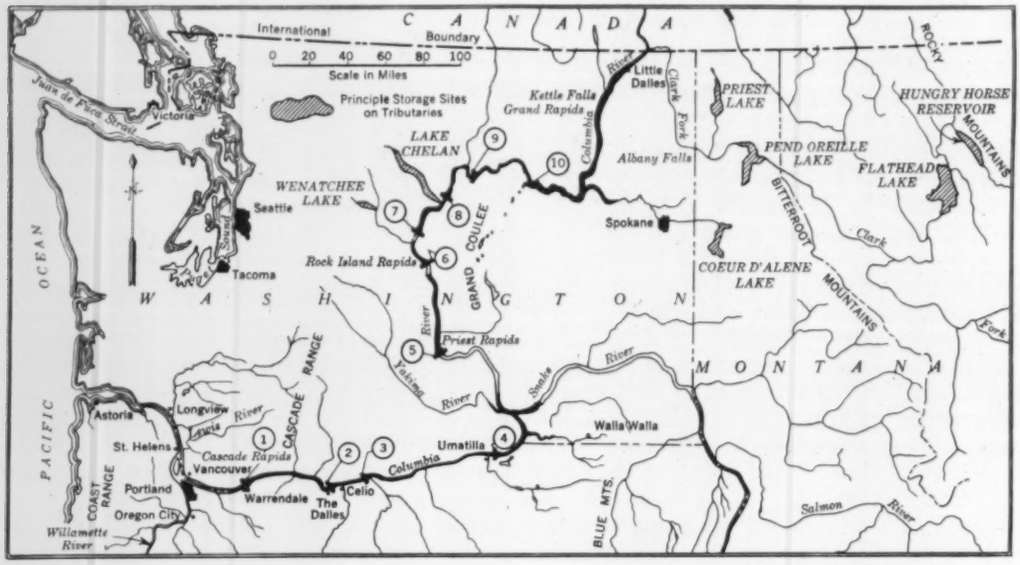


FIG. 1. PROPOSED PLAN OF DEVELOPMENT OF THE COLUMBIA RIVER BASIN
(1) Warrendale (2) The Dalles (3) John Day Rapids (4) Umatilla Rapids (5) Priest Rapids (6) Rock Island (constructed) (7) Rocky Reach (8) Chelan (9) Foster Creek (10 Grand Coulee)

All the dam sites listed in Table II have been investigated by eminent geologists, and the foundation conditions more or less explored by core drilling. Although there is no reason to believe that the engineering difficulties to be encountered at some of these sites cannot be surmounted, none the less these difficulties are not inconsiderable. The proposed dam at Warrendale involves a structure that must provide safely for a possible flood discharge of 1,400,000 sec-ft. and must be built on a foundation of sand and gravel not far downstream from a great slide that in comparatively recent times pushed the Columbia River out of its course and formed the Cascade Rapids and the pool in the river between these rapids and The Dalles. There is no precedent for such a structure at a site with this type of geological formation. Likewise the dam proposed at The Dalles, which requires the closure of a channel 500 ft. wide and 150 ft. deep, passing large flood flows, involves methods of construction without precedent.

Available data are insufficient to permit final conclusions on the best designs and methods of construction, but it is believed that the studies undertaken give a reasonable basis for estimating the cost of the structures and the time needed for their building.

At the headwaters of the Columbia and its northern tributaries are a number of large lakes which act as natural detention reservoirs to equalize the flow, and which could be used for further regulation. In the studies that have been made, the future storage considered, in addition to that behind the Grand Coulee dam, includes about 14,000,000 acre-ft. on the upper Columbia and its tributaries, about 10,000,000 acre-ft. on the Snake watershed, and about 2,000,000 acre-ft. on tributaries of the main stream below the mouth of the Snake.

NEEDS OF NAVIGATION CONSIDERED

When the Pacific Northwest was first opened up, the Columbia and Snake rivers were the only trade routes. Below Vancouver, sea-going traffic has been continuous since 1792 and is a vital factor in the economic life of the country. Above tidewater, railroads and highways have for the time being largely supplanted the rivers as means of communication, but the Columbia still remains an important potential artery of commerce.

The Columbia River has been under improvement by the United States in the interest of navigation for many years. The bar channel has been stabilized and developed by extensive jetty construction, and now has a width of one-half mile and a depth of over 40 ft. at mean lower low water. Between Portland and the sea a ship channel 500 ft. wide and 35 ft. deep has been practically completed. Canals with locks affording a depth of 8 ft. have been constructed around the Cascade Rapids and the Dalles-Celilo Rapids. From Celilo Falls to the mouth of the Snake many obstructions have been removed from the channel, increasing somewhat the depth and lessening materially the danger to navigation. Above the mouth of the Snake, in the

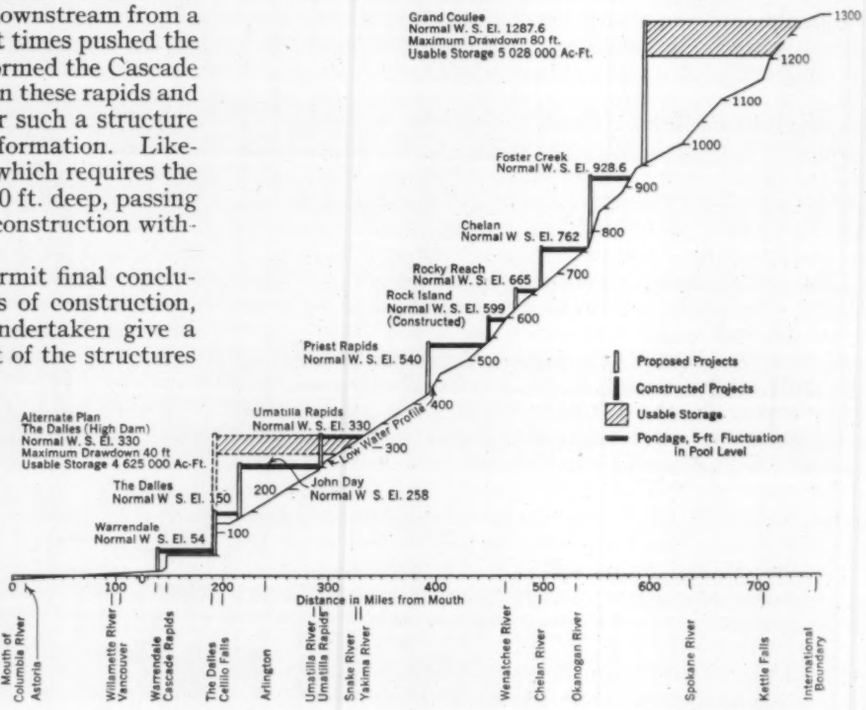


FIG. 2. PROFILE OF THE PROPOSED COLUMBIA RIVER DEVELOPMENT

240-mile reach extending to Kettle Falls, open river work has been prosecuted to obtain a depth of from 5 to 7 ft. Ships of all sizes and all nations now ply the lower Columbia, but notwithstanding the improvements that have been made, the non-tidal section of the river can be navigated only by high-powered boats of relatively small cargo capacity, and is unsuited for large tows.

Ocean-going commerce on the Columbia River now amounts yearly to over 6,000,000 tons, valued at over $300,000,000. In addition, river vessels on the tidal section handle each year about 2,000,000 tons of freight and 4,000,000 tons of rafted logs and piling. On the non-tidal section of the river the tonnage has always been relatively small.

Above the Snake River conditions are not favorable for the future development of any considerable amount of water-borne commerce, but in the non-tidal section below the Snake the situation is different. There is a large and relatively expensive movement of freight by rail parallel to the river in and out of the tributary area. A large percentage of the outbound freight is wheat, and of the inbound freight, petroleum products. These commodities could move by water at a substantial saving if the river were improved for efficient barge traffic. Studies made in this connection show that it is not unreasonable to estimate the prospective commerce on the middle section of the river at 600,000 tons per year, with a resultant total gross saving in transportation costs, including terminal charges and haulage to and from the river, of about $1,000,000 per year.

The effective method of improving this middle section of the river for navigation is in combination with the development of potential water power. The estimated cost of the locks and channel enlargement necessary for navigation in such a combined development is $16,100,000. The potential navigation is sufficient to justify the requirement that power developments on this stretch of the river be constructed so as to provide suitable pools for navigation, and the public benefits from navigation are sufficient to warrant the Federal Government in assuming the entire cost of the necessary locks and channel enlargement.

Above tidewater the Columbia flows through a deep and well defined channel and the area of adjacent lands overflowed is too small to warrant consideration. However, along the tidal section the valley floor includes considerable areas of fertile land subject to inundation, thus causing a flood control problem.

SITE FOR THE DALLES DAM, COLUMBIA RIVER

LEVEES PREFERRED FOR FLOOD CONTROL

Study of the various possible methods of flood control for lands along the tidal section of the Columbia River leads to the conclusion that leveeing is the most suitable method. It is considered that the parties interested would be justified in increasing the effectiveness of existing protection and extending it somewhat to unprotected lands, but that this is an independent local problem which cannot profitably be combined with navigation, power development, or irrigation. Some minor degree of flood protection for these lands will be an incidental result of storage developments.

Since stream flow is liberal and naturally regulated to a considerable degree, while the slopes of the streams are

TABLE II. PLANT CAPACITIES AND COSTS FOR TEN POWER SITES ON THE COLUMBIA RIVER

Site	Maximum Height of Dam	Useful Storage Thousands of Acre-Ft.	Flow 90 Per Cent of Time Thousands of Sec-Ft.	Corresponding Head In Ft.	Power‡ Capacity 90 Per Cent of Time Thousands of Kw.	Power Installation Thousands of Kw.	Construction Cost in Millions of Dollars§			
							Dam and Reservoir	Power Facilities	Total	Cost per Kw. Installed
Grand Coulee	475	5,028	45.0*	316†	970	1,575	138.74	32.45	171.19	$108.70
Foster Creek	205	36	28.4	164	317	635	24.52	19.95	44.47	70.00
Chelan	137	47	30.1	93	190	380	22.68	12.65	35.33	92.80
Rocky Reach	80	20	31.0	59	125	250	23.68	9.65	33.33	133.10
Rock Island	119	10	32.2	50	110	180	(60,000 kw. now installed)			
Priest Rapids	192	160	32.2	132	289	580	41.30	14.82	56.12	96.80
Umatilla Rapids	104	135	66.0	71	318	635	26.30	21.45	47.75	75.20
John Day Rapids	159	245	66.0	108	485	970	45.60	51.60	97.20	100.00
The Dalles (low dam)	260	80	70.7	96	462	925	22.05	41.75	63.80	69.00
Warrendale	80	110	74.0	52	262	525	24.10	25.80	49.90	95.10
					3,528	6,655				

* Based on a draw-down of 80 ft. at Grand Coulee. † Average head. ‡ (Flow 90 per cent of time) × (Corresponding head) × 0.068. § Excluding facilities for navigation.

PROPOSED LOCATION OF THE GRAND COULEE DAM AND PUMPING PLANT
To Develop Power and to Irrigate the Columbia Basin

relatively steep and include many sites where high dams can be constructed without extensive flowage damage, the Columbia River and its tributaries afford unusual opportunities for the development of great quantities of cheap hydro-electric power. It is estimated that 40 per cent of the potential water power of the United States is in the three states of Washington, Oregon, and Idaho. The rate of growth of installation of plants and the production of current in these states are shown in Fig. 3.

The quantity and cost of the power that can be developed at the ten proposed dams listed in Table II will vary with the dependable stream flow and the size of the installation. Based on the natural flow available for 90 per cent of the time, and plant capacities that could use twice that flow, the power installations at the nine dams below the Grand Coulee project would be as shown in the table. The installed capacity given for the Grand Coulee project is based on regulation from its own storage.

The costs per installed kilowatt that are shown in Table II indicate the relative cost of power from any one of these projects if constructed as a first step. The regulatory effect of the Grand Coulee project on the dams below, when and if they are constructed and can utilize added regulation, is an important asset. With its regulation, the power capacity of these nine projects is increased from 2,558,000 kw. to 3,500,000 kw., and their installed capacity, if twice the power capacity, is raised from 5,080,000 kw. to 7,000,000 kw.

With the added regulation from 14,000,000 acre-ft. of storage on the upper Columbia River and its tributaries, and 12,020,000 acre-ft. in the Snake River basin and the tributaries of the lower Columbia, after due allowance is made for future irrigation requirements, the power capacity of the ten projects would be about 5,212,000 kw., and the installed capacity about 9,685,000 kw.

It is estimated that power from a low dam at The Dalles, with financing at 6 per cent, could be delivered in Portland for 2.9 mills per kw-hr. if the plant were operated on a load factor of 55 per cent. On the basis of a normal price for fuel oil, and a load factor of 55 per cent, the cost of steam power at Portland or Seattle at the switchboard of large modern plants is estimated at 4.51 mills per kw-hr. This indicates that power from the Columbia River projects is cheap when compared with power generated by steam.

On the other hand, the great expenditure and the large blocks of power involved in constructing these hydro-electric projects, particularly the larger ones, is an unattractive feature to power companies unless Federal financing is authorized. For many years to come these companies can find sites on tributary streams that will furnish cheap power in blocks more suitable to their market requirements.

In forming an idea of the situation as a whole it should be kept in mind that the hydro-electric projects placed in operation in this market area since 1930, together with those for which plans are developed, will provide twice the amount of electric power that has been installed here from 1920 to 1930; also, that the estimated power installations of the ten projects of the proposed Columbia River plan, with regulation from the Grand Coulee project only, are equal to 12 times the electric power installed in the market area from 1920 to 1930, and amount to 25 per cent of the present electric power installations in the United States.

Over most of the Columbia watershed east of the Cascades where topographic features are favorable to agriculture, rainfall is insufficient for crops other than wheat and grass, and for wheat only in selected areas and by dry farming methods. Almost all the small tracts of good land to which water could be diverted from nearby streams by individuals were taken up between 1850 and 1890. Next came the development of low bench lands that could be watered by diversion through small canals, the construction of which was possible by local effort or corporate enterprise. Finally, after 1903, when the U.S. Bureau of Reclamation began to construct projects, irrigation was extended on a large scale. At the present time the total area under irrigation in the Columbia watershed is nearly 4,000,000 acres, and includes most of the land that can be developed at low cost.

The total amount of undeveloped

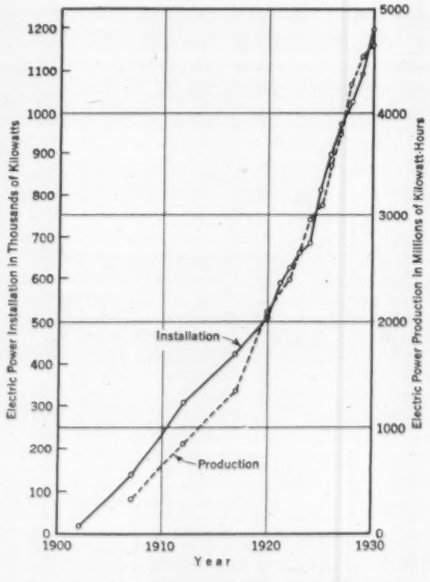


FIG. 3. GROWTH OF ELECTRIC POWER
In Washington, Oregon, and Idaho

land irrigable from the Columbia River proper is approximately 2,000,000 acres, about three-fourths of which lies above the mouth of the Snake and is mostly included in the so-called Columbia Basin Irrigation Project. The greater part of the large area of upland included in this project can be irrigated either by gravity diversion from Clark Fork and the Spokane River or by pumping from the Columbia River, using secondary power developed at the proposed Grand Coulee dam. As to the irrigable land below the mouth of the Snake, the studies show that the most feasible plan of improvement is by pumping from the Columbia River in connection with power development. The Columbia Basin Irrigation Project includes the largest and finest body of unreclaimed land left in the region.

HEAD OF THE GRAND COULEE
Looking Northeast up the Columbia River Valley

Preliminary plans and estimates for irrigating the remaining undeveloped land along the Columbia River have been made. A summary of costs and annual carrying charges for the Columbia Basin Irrigation Project, and for the tracts of irrigable land below the mouth of the Snake is given in Table III.

COLUMBIA BASIN PROJECT NOT JUSTIFIED

These data do not include any part of the cost of power dams. It is assumed that the interest rate will be 4 per cent, and that repayment will be made within 40 years on an amortized basis, and that the rate of settlement will be 50,000 acres of irrigated land per year. The cost of electrical energy from proposed power dams for pumping has been assumed at ¾ mill per kw-hr. above the Snake River, and at 1 mill per kw-hr. below it.

Studies have also been made to determine the permissible charge to the farmer for water. After consideration in detail of the probable market conditions and prospective demand for the various crops that can be grown, distribution of crop acreages, return per acre, and cost of production, it is not believed that the farmer on any of the tracts in question can make a living if he has to pay more than $6.00 an acre per year for water. This is considerably higher than the average charge made by the U.S. Bureau of Reclamation, but is approximately equal to the average charge in Western irrigation districts.

From the foregoing it will be seen that the cost of the gravity plan for the Columbia Basin irrigation project is considerably more than the cost of the pumping plan. Also that, as a business proposition, irrigation of land from the Columbia River is not feasible at this time. It has been proposed by various interests that irrigation in the Columbia Basin be subsidized by the profits from the sale of power developed by the Federal Government at one or more of the proposed dams in the main stream. This could be done if power in sufficient quantities could be sold at a high enough rate. However, in view of the existing world-wide over-production of agricultural products and of the extensive and long-continued depression, particularly in the farming industry, there does not appear to be any justification for making large outlays from the national treasury at this time in order to bring more land under cultivation.

The investigations and surveys of the Columbia River were carried out under the supervision of G. R. Lukesh, M. Am. Soc. C.E., Colonel, Corps of Engineers, Division Engineer, North Pacific Division. They were made by the Seattle Engineer District, of which John S. Butler, M. Am. Soc. C.E., Major, Corps of Engineers, is District Engineer; and by the Portland Engineer District, of which Major Oscar O. Kuentz is District Engineer. To the Seattle District was allocated the section of the river above the mouth of the Snake, and to the Portland District the section below. The reports on these investigations and surveys have been completed and transmitted to Congress. They will be published soon, probably as House Document No. 286, 72d Congress, First Session.

TABLE III. IRRIGABLE AREAS, COSTS, AND ANNUAL CHARGES
In the Columbia Basin and the Territory Below the Snake River

Tract	Area in Acres	Construction Costs: Total in Thousands of Dollars	Construction Costs: Per Acre	Annual Costs per Acre: Interest and Repayment	Annual Costs per Acre: Pumping from River	Annual Costs per Acre: Maintenance and Operation	Annual Costs per Acre: Total
Columbia Basin Project:							
Gravity Plan	1,519,890	598,285	$393.63	$19.89	$....	$3.13	$23.02
Pumping Plan	1,174,630	207,397	176.56	8.92	1.28	3.00	13.20
Cold Springs	50,000	7,975	159.60	8.06	4.13	3.00	15.19
Horse Heaven	142,000	20,598	145.07	7.33	2.80	3.00	13.13
Willow Creek	173,060	23,230	134.23	6.78	3.07	3.00	12.85
Arlington	11,500	1,427	124.07	6.27	2.73	3.00	12.00
Roosevelt	1,500	170	113.37	5.73	0.92	3.00	9.65

Construction Methods on the Cle Elum Dam

Difficulties of Tunneling in Soft Ground and Other Problems Successfully Met

By W. H. GARDINER

DIRECTOR, WINSTON BROTHERS COMPANY, RONALD, WASH.

SEVERAL construction features serve to make notable, from the standpoint of the engineer, the work on the Cle Elum Dam, now under way in the Yakima Valley in Washington. The structure is of considerable size, including over 1,000,000 cu. yd. of embankment, 21,000 cu. yd. of excavation in tunnels and shafts, and 33,000 cu. yd. of concrete. Of outstanding interest were the difficulties encountered in tunneling and shaft-sinking in the soft ground. The solution of these problems taxed the resources of the contractors, as Mr. Gardiner shows in this article, covering operations up to the summer of 1932. This account is an abstract of his full paper, delivered on July 7, 1932, before the Construction Division at the Convention of the Society in Yellowstone National Park.

THE Cle Elum Dam is to provide an enlarged reservoir for the Yakima Irrigation Project, serving the irrigated lands in the fertile and populous Yakima Valley, mainly between Yakima and Kennewick, Wash. It crosses the Cle Elum River at the lower end of Lake Cle Elum, displacing an old timber crib dam which was capable of raising the lake level 10 ft. The proposed dam will raise the water 110 ft., so that the combined raise in dam height and lowered outlet level will provide 420,000 acre-ft. of storage when the spillway gates are ultimately installed—an increase of 394,000 acre-ft. This will enable the 300,000 acres of lands already supplied with canals and ditches to receive one complete additional watering, and make possible the irrigation of a large added acreage.

According to geologists, a glacier deposited a gravel and clay barrier reaching across the full width of the valley, forming a natural dam for the lake left by the glacier when it receded. The restoration of the original barrier across the river channel will be accomplished by the building of the dam. The lake is held at its present level by a clay-plaster of glacial origin, which has been washed against, and has worked its way well into, the lake face of this natural barrier, thus helping to make it water-tight. The character of the material to be placed in the dam will render it impervious, and the earth abutments on either side will be heavily reinforced with impervious earth blankets. The rock walls of the valley are at least a mile distant on each side, and the lake is very deep, containing many areas locally reputed to be bottomless.

The extent of the work may be visualized from Figs. 1 and 2. In reality there were three major items of construction: the earthwork of the main dam and dike embankments; the outlet diversion and control tunnel, including the shaft and operating machinery; and the spillway, together with the stilling basin and the deepening and enlarging of the river channel downstream. It may be interesting to note that the cost of the main dam embankment forms only 25 per cent of the total cost.

The award of the contract for the dam and appurtenant works was made to the Winston Brothers Company on August 15, 1931, and work was immediately commenced. The total amount of the contract will eventually be about $1,400,000, and the estimated value of the materials to be furnished by the Government is $350,000, making a total of $1,750,000 for the dam and appurtenant works. Additional past, current, and future expenditures by the Government for right-of-way; clearing of the reservoir site; road work; spillway gates; and various overhead items, will make the reservoir project cost an aggregate of $2,730,000.

Work in connection with

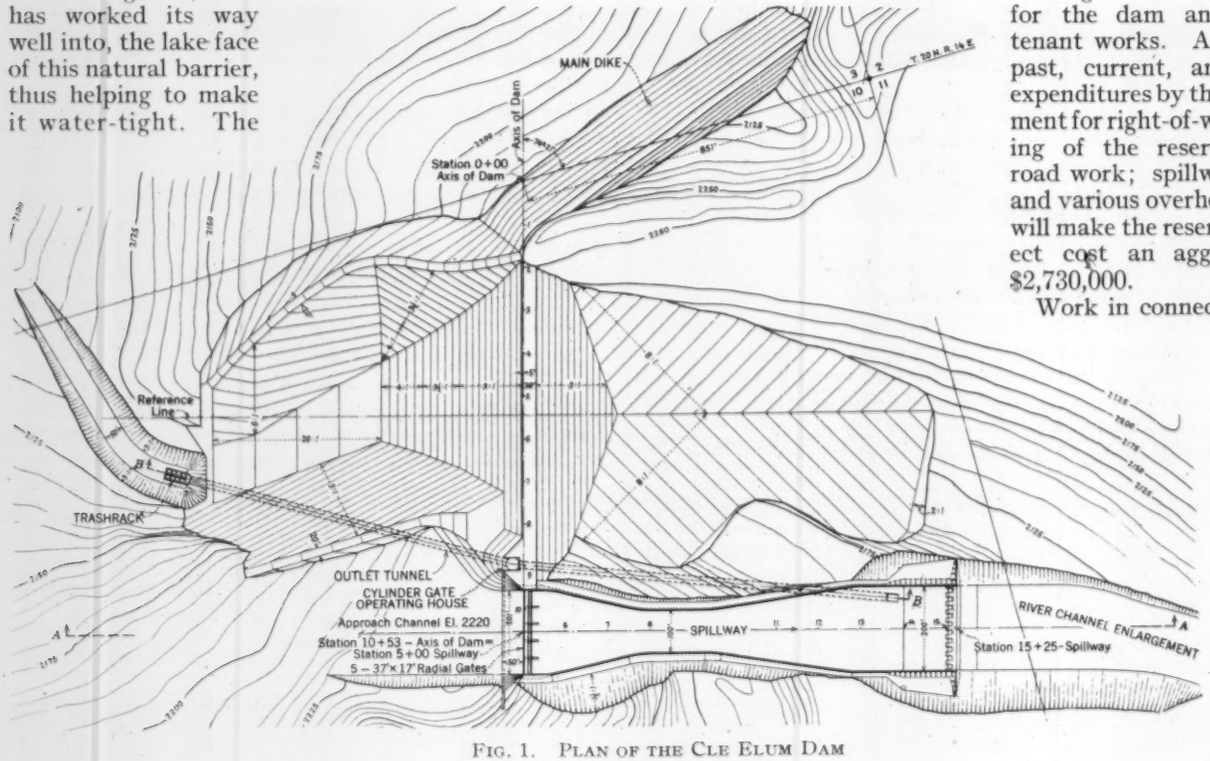


FIG. 1. PLAN OF THE CLE ELUM DAM

Sections Are Given in Fig. 2

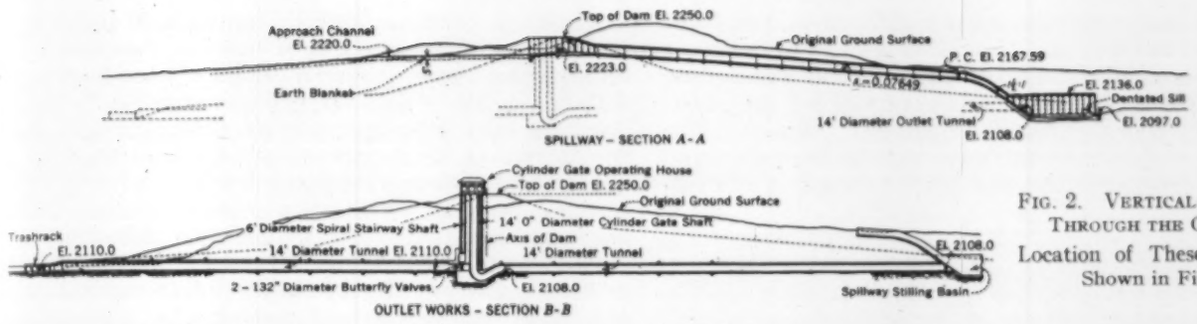


FIG. 2. VERTICAL SECTIONS THROUGH THE OUTLET; Location of These Sections Shown in Fig. 1

the excavation and lining of the tunnel and gate shaft, which has been completed, probably forms the most interesting and difficult part of the entire construction. The plans specify that the discharge end of the diversion tunnel be connected to the upper end of the stilling basin. Yet it was not possible to complete the work on the stilling basin and that of deepening and enlarging the channel below in time to divert water through the tunnel and into the stilling basin by June 1932. To excavate the stilling basin and not haul the excavated material directly into the main dam would have meant stockpiling, and rehandling at a prohibitive cost, several hundred thousand cubic yards of earthwork for which there was no practical storage space. The contractor therefore developed a plan of diverting the water back into the original river channel through a short section of temporary approach channel and tunnel branching off from the lower end of the main outlet tunnel, thus bypassing the water around the stilling basin. This temporary adit tunnel and channel are to be discontinued and closed after the completion of the stilling basin.

PROBLEMS OF TUNNEL EXCAVATION

Excavation of the tunnel proceeded to the gate shaft from two headings, the upper, or lake portal, and the lower, or adit portal. Steel liner plates, or steel ribs if needed, had to be used to lag the tunnel and gate-shaft excavation; timbering was not permitted except for lagging the temporary adit tunnel, of which the inner half was required to be concrete lined. The main outlet tunnel was increased at the time of driving to 19 ft. in diameter. In fitting the rings of steel liner plates together, the flange joints were staggered for additional rigidity in maintaining the form of a true circle.

This ring, or shield, was carried flush with the face of the excavation so that no more roof was ever uncovered than was sufficient for the bolting up of one liner plate at a time, and in this way the driving was accomplished without any slips or loss of ground in the gravel roof above the lagging. Voids behind the plates were tightly packed with pea gravel as the driving proceeded. The top of the ring was usually carried about 2 in. higher than a true circle and the sides drawn in about 1 in., the roof load flattening the ring so that the radii were equal. Temporary timber posts were carried along behind the driving at 10-ft. intervals to provide a rigid support for the roof plates until uniform settling had taken place, thus preventing flattening and distortion of the ring.

It was customary, in placing the liner plates, to start at the top of the circle and carry down with one ring and then work upward with two rings at the same time. In this way it was quite easy to carry the line and grade with exactness. As it was entirely below the level of the river, the tunnel was expected to be very wet, but the section between the lake portal and the gate shaft proved to be surprisingly dry, largely due to the considerable amount of clay with which the gravel was impregnated. Only one 4-in. pump, working a small part of the time, was required at the upper portal sump to keep the tunnel unwatered. The lower or adit end was full of water courses, and there it was necessary during most of the excavation to use two 6-in. pumps continuously.

Progress in driving varied in accordance with the looseness of the gravel formation. At the junction of

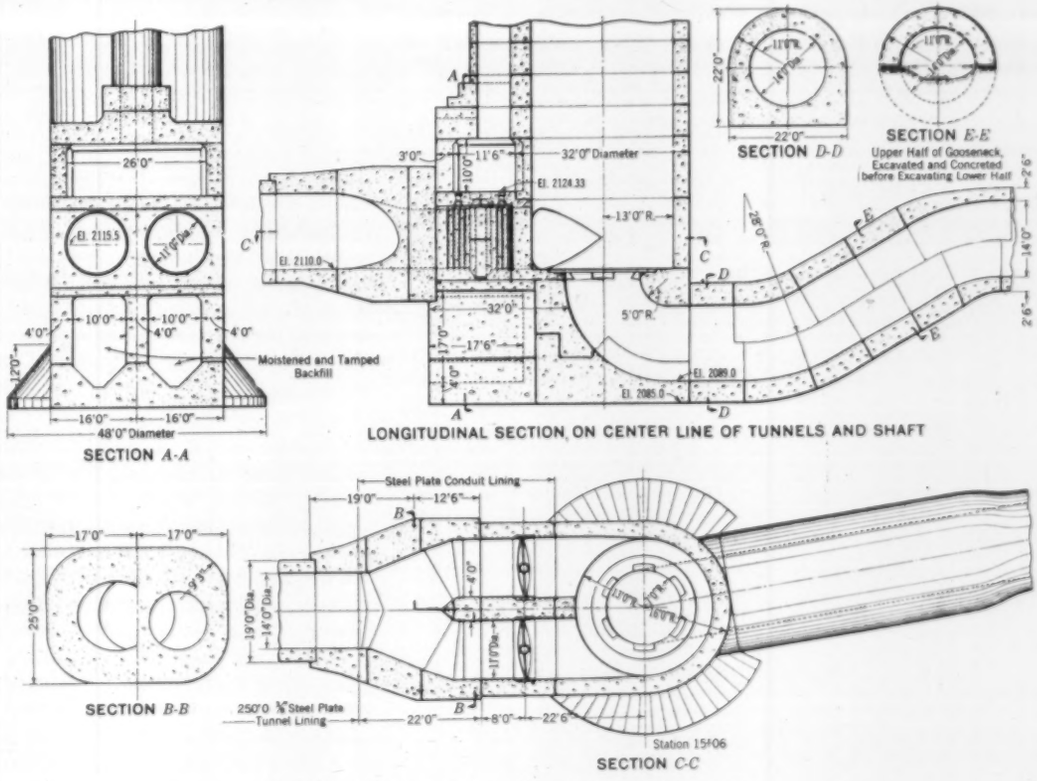
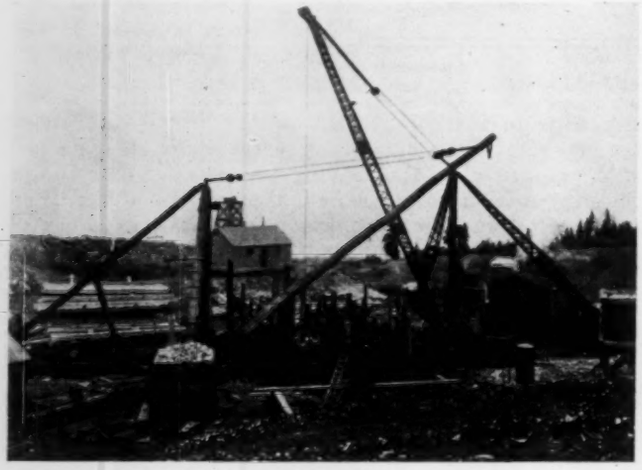


FIG. 3. DETAILS OF THE GATE SHAFT, LOWER END

the adit tunnel with the main tunnel the ground was very loose, and progress sometimes averaged no more than 1 ft. per 8-hour shift. Where the formation was tighter, the adit tunnel progressed at the rate of 3 ft. per shift; and in the lake section, as fast as 4 ft. per shift. The material was broken down from the face either with hand picks or air spades. No powder was used, and as a rule



STIFF-LEG DERRICKS OPERATING AT GATE-CONTROL SHAFT
During Excavation and Driving of Sheet Piles

great care was required to prevent slips. Muck was loaded into 1-cu. yd mine cars by hand or by an electrically-operated half-yard scraper loading machine, and hauled to the portal by gas dinkey or, where the haul was over 500 ft., by an electric storage-battery locomotive to avoid danger from exhaust gases. Thence it was moved up the portal approach ramp by electric tugger hoist, and out to the dump by gas dinkey.

The work progressed satisfactorily and entirely without accident, either to the work or to the men, with the result that there were no gaps to be filled later on with grout, other than plugging up the open water courses at whatever level they were encountered. Natural ventilation was all that was required. The progress of the tunnel work, both excavation and concreting, is given in Table I.

CENTRAL MIXING FOR TUNNEL LINING

Lining of the tunnel with concrete was commenced almost as soon as the driving in the lake end of the main tunnel was completed. The concrete was mixed in a central plant about midway between the two portals. Adjacent to the mixing plant were stock piles of three sizes of concrete aggregate, the material being dumped from an overhead trestle after being truck-hauled from the unloading point $2^1/_2$ miles away. The aggregate was withdrawn from the stock piles by a belt conveyor in a trap tunnel underneath the piles, which elevated it to a three-compartment bin at the top of the mixing plant. Sacked cement was likewise truck-hauled from the rail delivery point. Storage for eight carloads was provided in a house adjacent to the mixing plant, whence it was platform-trucked directly to the batch-hopper.

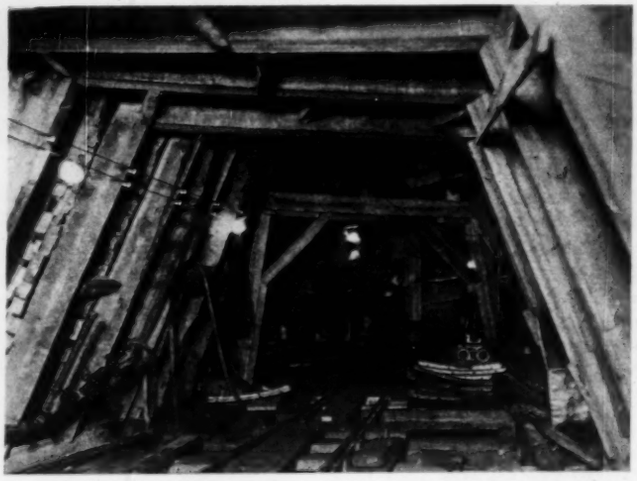
TABLE I. PROGRESS OF TUNNEL WORK, 1931–1932

WORK	BEGAN	ENDED	DAYS	LIN. FT.	CU. YD. PER LIN. FT.
Tunnel Excavation:					
Lake End	Oct. 11	Dec. 14	65	530	10.6
Adit End	Nov. 2	Feb. 17	107	1,003	10.6
Tunnel Concrete:					
Lake End	Jan. 19	Feb. 27	40	524	4.8
Adit End	Mar. 3	Apr. 18	45	903	4.8

The diameter of the finished tunnel was 14 ft., so that the concrete lining was $2^1/_2$ ft. thick. This was reinforced with an inner and an outer set of hoop bars, each with longitudinal bars. As further protection, the inner 250 ft. of the tunnel, upstream from the gate shaft, and the splitter of the transition section were lined with welded steel plate to a thickness of $^3/_8$ in.

The entire concrete lining was placed pneumatically in one continuous operation for a section 20 ft. long, one section being placed every 24 hours. The customary procedure was for the day shift to place the concrete, which took practically the entire eight hours for the 95 yd. The second, or "swing" shift permanently placed the outside ring of reinforcement and temporarily placed the inside ring, hanging the upper bars to the bars of the outer ring; it also cleaned up ahead of the next pour. The third, or "graveyard" shift set the forms, completed the inner ring of steel reinforcement placing, and stripped the forms from the concrete poured two days previously.

Three sets of wooden forms, metal-faced, were used.



HEADING SECTION AT INNER END OF ADIT
Looking Into Junction Chamber at Transition Connection with Main Diversion Tunnel. Note Skewed Collar Sets and Rise in Roof

These consisted of 20 segments to make up the circle 14 ft. in diameter. Each segment or panel weighed approximately 350 lb. These were moved ahead from the point of stripping to the point of the next set-up by means of a temporary hanger and trolley suspended from tunnel liner plates and forms. During the placing of the concrete in the side walls, the discharge hose was shifted from one side to the other to keep the levels equal and thus avoid excessive thrust on one side.

CONSTRUCTION METHODS DEVELOPED

Since practically all the tunnel concrete was placed during winter weather, the materials were pre-heated. Jets of live steam were introduced into the aggregate stock piles through the trap tunnel, and were run up into the aggregate from the bottom of the hopper bins over the mixer. The batch water was also heated with live steam. The concrete was brought into the forms at a temperature not exceeding about 70 deg. fahr. to avoid too rapid setting while placing. No trouble was encountered in securing excellent results for the entire section poured.

Concreting was started at the inner end of the lake section, and continued outward toward the portal. By the

time it was finished in this 524-ft. tunnel, excavation had been completed at the adit end, to which the pneumatic concrete placing equipment was therefore transferred.

Cut-off collars were added to the tunnel lining at intervals, both in the open cut and the underground sections, to reduce the velocity of any water tending to flow alongside, between the metal lagging and the natural ground. The main tunnel was completely driven before the underground collars were enlarged, which was done by removing two rings of liner plates. Except for one collar, placed ahead of the lining, the concrete was gunned integrally with the lining. The collars were reinforced with two grillages of bars which projected and were hooked into the main tunnel concrete.

STEEL PLATE LINING USED

The steel plates for the 250 ft. of tunnel that had to be lined were delivered bent to the proper curvature in 100-in. lengths, three being required for a circle 14 ft. in diameter. They were assembled and welded in 25-ft.

SHOVEL REMOVING OLD CRIB DAM AT DAM SITE WITHOUT DISTURBING INTAKE OF CLE ELUM GRAVITY WATER SUPPLY
Logs Cleared from Reservoir Site Boomed and Sold

lengths of completed conduit just outside the lake portal, lowered to a car truck on the tunnel tracks, and hauled to the point of installation. They were then jacked into position, and the final circumferential joint between 25-ft. sections was welded. Each section served as an inner concrete form, and 25 ft. of lining was placed immediately following its installation. The inner surface of the metal lining was painted with three coats of water-gas-tar paint.

The tunnel intake is a reinforced concrete trash-rack structure having on its end, sides, and top a total of 18 openings 10 ft. square, in which are placed $^{7}/_{8}$ by 6-in. steel bars on 6-in. centers. This intake is joined with the underground tunnel section by 212 ft. of circular, reinforced concrete conduit 14 ft. in diameter and 27 in. thick. Between the trash-rack structure and the upper end of the tunnel, 12-in. stop logs can be placed to completely close the tunnel. The inside conduit forms were those used in the tunnel section. They were held down against flotation by anchor bolts set in saddle blocks previously poured.

Pressure grouting for the sealing of voids and waterways behind the tunnel lining was started as soon as practicable after the tunnel had been concreted. Two 2-cu. ft. pneumatic grouting machines were used, operating with a pressure of 50 lb. per sq. in., and placing a mixture of one part sand to one of cement. About 33,000 cu. ft. of grout were placed before all seepage was cut off.

Exceeding the tunnel work in interest and requiring even more care, were the operations of excavating and concreting the lower levels of the gate shaft and the tunnel transition section leading into the shaft (Fig. 3). At the bottom of this shaft are two 11-ft. emergency butterfly gates, electrically and manually controlled, set

DRY ROCK PAVING AROUND TRASH-RACK STRUCTURE
Openings in Top, Sides and End; Bars on 6-In. Centers

side by side and at a considerable angle to the main shaft, and a pair of 18-ft. tandem cylinder gates operated by the control machinery at the top of the shaft. The specifications provided for payment for excavation only within the neat section of concrete, assuming perhaps that the contractor would first drive the circular cylinder gate shaft and adjoining stairway well, and then undercut and underpin the excavation for the butterfly valve chamber.

Being entirely in gravel formation, this operation of underpinning would have involved so much roof support that getting the butterfly valves and conduit-lining sections into place by lowering them down the cylindrical gate shaft, and side jacking, would be next to impossible. In any event it would have been a far more hazardous and costly operation than sinking the shaft for its whole depth in the square and half-circle section existing at the butterfly valve chamber. Hence a shaft 48 ft. long by 32 ft. wide, one end semicircular, was driven from the surface of the ground to a depth of 140 ft.

To meet the requirements as to metal lining of the excavation, it was finally decided to drive a cell of interlocking steel sheet piling around the outside perimeter of the excavation for its entire depth, braced with 12 by 12-in. timbers, leaving wells of proper size for lowering the butterfly valves and metal work. The piling was left in place but the timber bracing was pulled out as the shaft was concreted. The space inside the cell, to the concrete around the stairwell, was backfilled with earth after the forms were stripped. The 16-in. sheet piling weighed 26.2 lb. per ft. of piling, and 17,200 lin. ft. were required. A length of 32 ft. of shaft was sunk in open cut before lagging was started. The excavation was kept from 2 to 4 ft. below the cutting edge of the sheet piling so that this edge was never actually driven into the gravel formation.

SHAFT TIMBERED AND CEMENTED

Near the top of the shaft, the spacing of the 12 by 12-in. timber sets was 9 ft. vertically, decreasing with increase

in depth and pressure to about 6 ft. at the bottom. The tendency was for the lower lengths to carry down with the one being driven, while the follower piles would fail to move. In many cases these joints had to be welded to prevent them from opening up and allowing ground slips. Since the driving became harder as more followers were added, shorter and shorter lengths of piling were used to prevent buckling at the start of the driving, the final sections being between 6 and 15 ft. long.

About 10 ft. above the bottom, a stratum of fine sand carrying a great deal of water was encountered. From this point down, the work was very difficult and expensive, and continuous pumping of large quantities of water was required. At times as much as 1,400 gal. per min. was pumped to the surface 140 ft. above. The driving was exceedingly hard, and when bottom was finally reached, the top of the piling was so badly battered that it would probably have been impossible to drive it to any greater depth, due to the extreme side pressure at the bottom.

When the bottom had been reached with this straight-cell section, a bell flare was excavated—a slow operation in cramped quarters—involving pole board driving of two sets of steel channel lagging, braced temporarily with timber, which was replaced by 8 and 10-in. I-beams and batter posts.

Concrete was carried up to the lower level of the transition section leading out to the lake section of the tunnel, and then excavation was started on this, which was perhaps the most tedious part of the job. At its junction with the gate shaft this transition was 34 ft. wide by 25 ft. high, and this section was carried out $12\frac{1}{2}$ ft. from the shaft before it began to diminish. Then, in a length of $15\frac{1}{4}$ ft., it was diminished to the dimensions of the main tunnel section. The support of a flat roof almost 34 ft. wide, under nearly 100 ft. of gravel—utilizing a metal lining while introducing sections of metal conduit and reinforcing steel prior to placing concrete—was an operation calling for all the experience possible in soft-ground work. Fortunately, there was no more than the expected amount of roof settlement and no ground slips during all the time of excavation and concreting, and no accidents were recorded.

Perhaps the most difficult part of the entire job was placing the splitter section of the conduit liner breeching and the reinforcement around it. No one was sorry when the concrete in this transition was brought out to the face of the gate shaft. This experience further emphasized the satisfaction felt in the decision not to underpin the butterfly valve chamber during excavation. We particularly appreciated the consideration shown by the engineers and inspectors in working with us in these difficult places, and in designing over and over again the reinforcement and concrete mixtures to fit in between the necessary supports for the roof and forms. Their experience and knowledge of conditions prevented a hard job from becoming what might readily have been an impossible one.

GOOSENECK SECTION ALSO AWKWARD

No sooner was the problem of this transition solved than that of the gooseneck section of tunnel came up. This section also was driven from the gate shaft. In this instance we were favored by having a circular arch instead of a flat roof over a large span. On the other hand, we were confronted with a bottom of wet, fine sand and a steep slope coming up to meet the main tunnel. It was considered impossible to excavate the entire circular section in one operation and provide sufficient bracing in the lining to resist the top and side pressure of the ground above. Consequently it was decided to drive and concrete the 10 ft. of straightaway section leading from the bottom of the gate shaft, and then to drive and concrete the top half of the circle of the remainder of the gooseneck, thus providing an arched support while excavating and underpinning the lower half in short sections with concrete.

As the horizontal section of the main tunnel had been completely driven and concreted at the upper end of the gooseneck, it provided a solid support for the arch at this end. No especial difficulty was encountered, except with the fine sand and water at the bottom, which made great care necessary to prevent the side-wall earth from being lost by washing out. It was necessary to provide an unusual amount of drainage with clay tile to keep the foundation dry for concreting.

Reinforcement in the shaft, transition section, and gooseneck was very heavy. In the transition section corrugated bars $1\frac{1}{2}$ in. sq. and closely spaced were used. They were welded as well as lapped, and it was next to impossible to get them in place past the temporary and permanent roof supports and metal conduit liner plates.

The concrete was volumetrically graded in the proportion of 1 part of cement to 2.4 parts sand and 3.6 gravel, with an admixture of dicalite amounting to $2\frac{1}{2}$ per cent by weight of the cement. The required strength of the concrete to meet the design specifications was 3,500 lb. per sq. in. after 28 days. To allow for differences between field and laboratory concrete, it was believed the cylinders should withstand 4,000 lb. under test. All tests met this requirement, and the average of the cylinders was practically 4,500 lb.

PROGRAM FOR COMPLETION

During the spring of 1932, between the advent of warm weather and the time for diverting the water from the main channel entirely through the outlet tunnel, a certain amount of grading work was possible. The main dike was completed, with the exception of the gravel blanket and riprap protection, and during this operation it was possible to acquire experience in the work that might be required on the main dam embankment. Material was excavated from the adjoining borrow pit and truck-hauled to the main dike embankment. It was then spread, rolled, and tamped as required. Tests in cross-section measurements reveal a compaction of approximately 12 per cent in material, and tests of the material in the borrow pit reveal weights of from 100 to 120 lb. per cu. ft., while in the compacted embankment the weights range from 127 to 130 lb. per cu. ft. As of August 15, 1932, about 275,000 cu. yd. had been excavated from the stilling basin and channel enlargement and had been hauled into the pervious section of the dam and the gravel and cobble blanket. In addition, approximately 250,000 cu. yd. had been excavated from the spillway and borrow pits and hauled into the impervious section of the dam. The completion of the spillway and main dam will be accomplished in 1933.

No suspense can be keener than that felt by those responsible for the lives of men and the safety of work and plant when high water suddenly threatens construction operations in the bottom of a swift and powerful river. Nor do I know of any more thrilling occupation than competition with the sometimes irresistible forces of nature when working against time to get a dam built. Very few things in a contractor's life afford more pleasure and satisfaction than working under a capable and cooperative engineer, such as is generally to be found in charge of dam construction work and as it is our good fortune to meet with on the Cle Elum job.

Construction Equipment for Hoover Dam

Solution of Unprecedented Problems Clears the Way for Record Accomplishments

By NORMAN S. GALLISON

DIRECTOR OF PUBLIC AND PRESS RELATIONS, SIX COMPANIES INC., BOULDER CITY, NEV.

THE very magnitude of the construction work for Hoover Dam introduced unheard-of engineering problems. Those concerned with the choice and operation of contractor's equipment are treated here. Only the railroad approach facilities had already been provided by the Government when the contractors, Six Companies Inc., reached the scene in March 1931. Their problems involved the securing of access to the site and the necessity of prosecuting the work at record speed. How well they overcame the difficulties of excavation, transportation, concrete manufacture, and the numberless coordinate undertakings is related in a most interesting manner by Mr. Gallison. His complete paper, here abstracted, was presented, including several reels of progress motion pictures, before the Construction Division at the Yellowstone Convention on July 7, 1932. Construction pictures also appear on page 1 at the beginning of this issue.

IN PLANNING for the construction of Hoover Dam, the contractors, Six Companies Inc., were confronted with four major considerations distinguishing this job from similar undertakings with regard to the selection of construction equipment: (1) the inaccessibility of the site; (2) the extreme climatic conditions; (3) the large tonnages involved; and (4) the short period allowed for completion.

The site selected for Hoover Dam in Black Canyon is approximately 30 miles from Las Vegas, Nev., the nearest railroad station. The topography of the country is extremely rugged; approximately a mile upstream from the site the river enters a precipitous, steep-walled canyon, which was impossible of access except by small boats.

Prior to the arrival, early in March 1931, of Six Companies Inc., the U.S. Government had done considerable preliminary work, including the building of a 22.5-mile standard-gage railroad from Las Vegas to Boulder City. Contracts had been let for the construction of a highway and a railway from Boulder City to points near the site. At the time Six Companies Inc. commenced operations, the only means of access to the vicinity was a rough, ungraded desert road leading to a point at river level several miles above the dam site.

An early decision as to permanent means of transportation had to be made in order to coordinate the roads needed for preliminary construction with those required later for muck disposal, delivery of materials, securing of aggregates for concrete, and transportation of workmen from Boulder City.

CLIMATIC AND OTHER CONDITIONS TO BE MET

During 1931, a maximum temperature of 136 deg. was registered in the canyon on July 25, and a minimum of 12 deg. on November 25. The effect of high temperatures on both machinery and men, and the necessity for uninterrupted operation during the summer, were constantly borne in mind in the selection of equipment.

Almost all items in the Hoover Dam contract were computed in millions of tons. Consideration had to be given, not only to the size of the equipment necessary, but also to its ability to handle large tonnages for protracted periods. Much of it is of special design with respect to size, and almost all the standard units are of the largest size manufactured.

The time element entered into the situation in two ways: (1) because of the immense amount of material involved, equipment had to be extremely rugged and relatively long-lived; and (2) on the other hand, its ability to perform the allotted task, without replacement and according to a rigid time schedule, was of prime importance.

The job as a whole naturally fell into several distinct divisions. These were studied separately, and special engineering reports were prepared. The construction method had first to be determined—considering general problems as well as cost (both initial and operating), simplicity, salvage values, and coordination with other operations—and then thought was given to the actual construction machinery to be used. Factors influencing the selection were: known reputation and performance of equipment gained from previous experience; ability of the manufacturers to meet unusual specifications and delivery requirements; and ruggedness, because of the expected heavy and unusual demands to be made upon it. In certain cases special consideration was given to mobility and portability, to electrical operation, and to the adaptability of the equipment to multiple-unit operation.

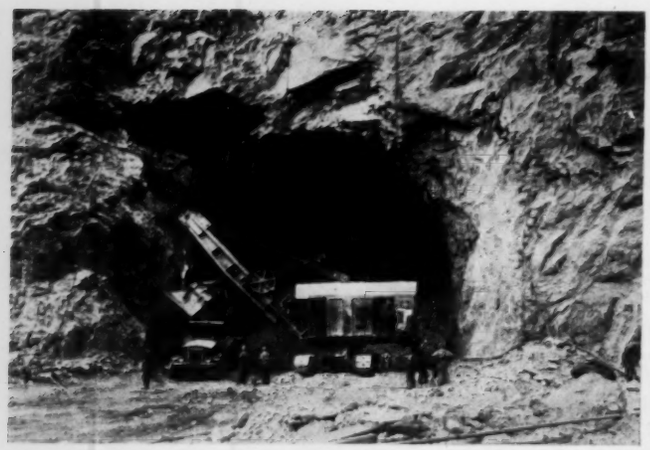
An attempt was made to standardize equipment as far as possible. In most cases it was decided that it would be advantageous to buy all equipment of a certain type from one manufacturer. Exclusive contracts for certain types of equipment, for the life of the job, were made with several manufacturers. This procedure was determined largely by a desire to reduce negotiations to a minimum, to centralize responsibility for installation, to keep the necessary stock of spare parts low, and to simplify and standardize the duties of operation, care, and maintenance. Other advantages gained were the interchangeability of spare parts among various units, and the ability to transfer workmen from one operation to another where the equipment was identical.

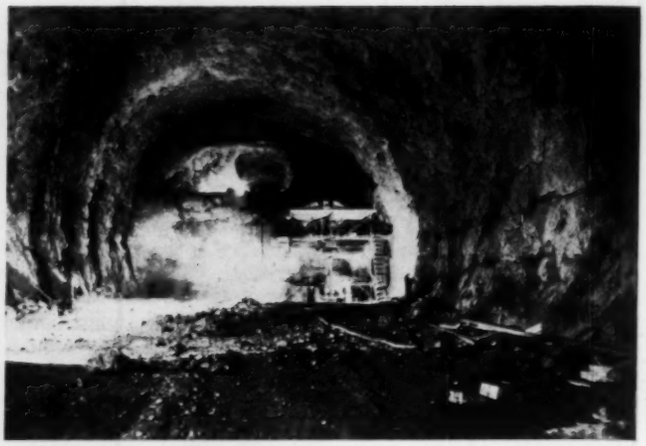
After careful study, it was decided to use a combination of transportation units—involving the construction of highways for the use of trucks, approximately 30 miles of railroad, and an extensive system of cableways—connecting the portal structures of the upper and lower diversion tunnels, and the dam, intake towers, and power house.

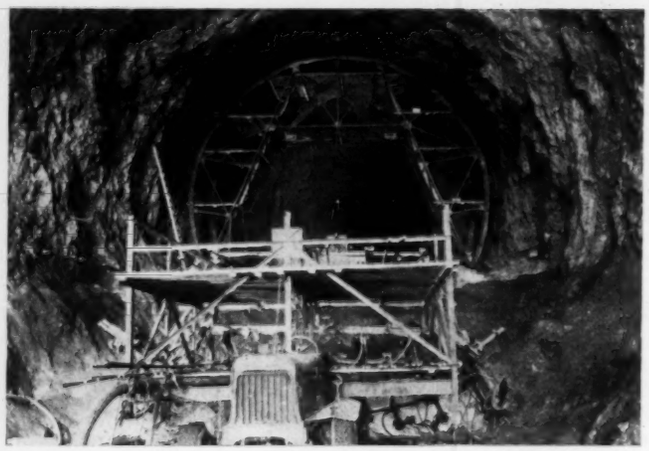
TRUCKS FOR MUCK DISPOSAL

Once the preliminary roads were cut through, the first major problem was the driving of the four main diversion tunnels, two on each side of the river, each 56 ft. in diameter and approximately 4,000 ft. long.

Since no waste material could be dumped into the river bed, it was necessary to find areas for disposal either in side canyons, which are few, or over the canyon rim, 800 to 1,000 ft. above the river. The latter method involved steep grades and sharp curves, and the volume of broken rock to be so disposed—about 2,000,000 cu. yd. in tunnels and approach cuts and an equal amount in the dam foundation, cofferdams, and miscellaneous struc-







DIGGING THE CIRCULAR DIVERSION TUNNELS, DRILLED AND SHOT IN FIVE STAGES

Left: 100-ton electric shovel loading dump truck at portal of one of the tunnels in the enlargement operation. Right: First stage, 12 by 12-ft. top heading; second stage, upper bench, 12 ft. high; third stage, main bench, 30 ft. high (drilling "jumbo" drills half the main bench face at one set-up). Below: Fourth stage, another drilling "jumbo" drills 14-ft. invert section; fifth stage, trimming "jumbo" in background scales projecting rocks to diameter of 56 ft.

tures—made it necessary to have several loading units working simultaneously and moving in different paths so as not to interfere with one another. The grades and curves could not be negotiated by railroad equipment, so recourse was had to the large modern truck to solve this transportation problem.

The trucks in use have body capacities of from 9 to 14 cu. yd., and when loaded weigh between 20 and 32 tons over all. Some are equipped with dual rear axles, and all have dual rear wheels, with the exception of the largest size, which uses two 40 by 14-in. trimmed solid tires on each rear wheel. A total of 160 pieces of motive equipment is in use on the job, of which 70 have capacities of 5 tons or over.

Since no manufacturer could furnish equipment having all the features desired, special bodies for handling rock were designed by engineers of the Six Companies Inc. Tramways could never have been successfully used to handle such large blocks of material as were excavated. The alternative of driving a long tunnel with a gentle gradient, from a more accessible area, was decided against, principally on account of cost, but also because of the probability of traffic congestion and the length of time that would be required to gain access to the canyon.

NOVEL EXCAVATION PLAN

After 12 by 12-ft. pilot tunnels had been driven and completed by the usual methods, a comparatively novel apparatus was developed for drilling the four diversion tunnels. This consisted of a large drilling "jumbo" mounted on a 5-ton truck chassis and equipped with 30 pneumatic drills, capable of simultaneous operation.

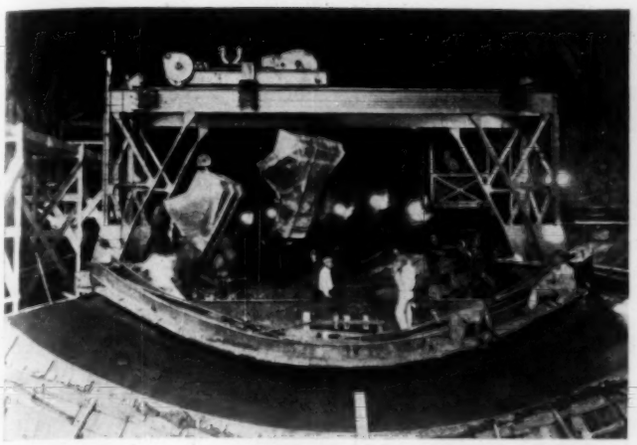
In enlarging the tunnel, at least 40 ft. of working headroom was needed for the operation of the type of electric shovels selected, which made it necessary for the main bench to be 30 ft. high (in addition to the top heading of 12 ft.) and 56 ft. wide, the full diameter of the tunnel. Since mucking was to be done into trucks, a truck-mounted drill carriage offered the logical solution. After experimentation, such an apparatus was devised, capable of drilling one-half the main bench with one set-up and able to pass the shovel in the tunnel. This "jumbo" proved to be extremely satisfactory in operation and was later modified and used for drilling the invert section.

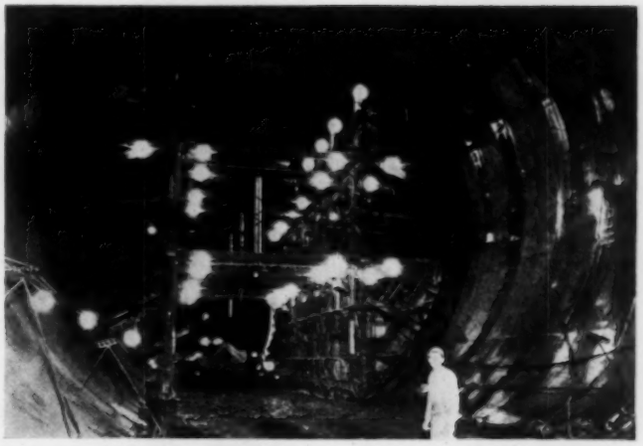
Muck was handled in the pilot tunnels by electrically driven mucking machines, which combined a conveyor belt with a digger dipper and were admirably suited for work in close quarters. These muckers dumped directly into 3-yd. mine dump cars, which were hauled by locomotives run by storage batteries. Drilling was accomplished by a small drilling "jumbo" mounted on a narrow-gage railroad car.

Considerable open-cut excavation was necessary in gaining access to the portals of the diversion tunnels for enlargement purposes. For this work nine electric shovels were used. They came to the job equipped with $2\frac{1}{2}$-yd. heavy rock dippers, but after some experience these were changed to dippers of $3\frac{1}{2}$-yd. capacity, somewhat lighter in weight, which proved quite successful.

Electric shovels possessed marked advantages for tunnel mucking for three reasons: their operation did not impose additional ventilation problems; they were obtainable in a size suitable for the economic prosecution of the task; and they effected remarkable operating economies. They have proved equally adaptable in the excavation of spillway open cuts, and when equipped with either dippers or draglines they will be used in the excavation for the cofferdam and dam foundation.

Over 16,000 cu. ft. of air per minute was needed to meet the peak requirements of the drilling schedule. Three compressor plants were constructed at strategic points and interconnected with 6-in. pipes. Compressors were installed with direct-connected synchronous motors, which acted as balancers on the line and equalized the pressure.





TUNNEL LINING 3 FT. THICK, POURED IN THREE STAGES

Left: Placing the invert. Traveling gantry crane carries two 2-cu. yd. buckets to point of placement. Right: Side walls poured simultaneously. Trucks haul 2-cu. yd. buckets from mixers. Crane on top of forms lifts buckets so that concrete can be chuted into place through ports in the steel forms. Below: Concrete of arch section placed pneumatically through 8-in. pipes by two cement guns. Arch has been coated with a bituminous curing paint.

Under the terms of the contract, the concrete in the dam must be artificially cooled to dispel the heat caused by hydration, and better to control the expansion and contraction of the mass. Hence it was necessary to select machines that would serve as efficient, reliable, and economical air compressors under rigorous conditions and which also could be readily converted into refrigerating units by the substitution of ammonia cylinders as needed.

LINING THE DIVERSION TUNNELS

In lining the diversion tunnels with an average thickness of 3 ft. of concrete, the method adopted by Six Companies Inc. was to divide the work into three stages: (1) pouring an invert section, comprising the lower arc of 32 ft., by means of a traveling gantry crane running on a track with a gage of 34 ft. 8 in., laid on a concrete bench, which became part of the invert lining; (2) pouring both side walls simultaneously up to within 10 ft. 9 in. of the top; and (3) pouring the arch.

Both the side-wall and arch forms were of fabricated steel construction, each 80-ft. form consisting of two 10-ft. and three 20-ft. sections bolted together. These forms were designed by engineers of the Six Companies Inc., and are collapsible and movable on the tracks laid on the concrete benches of the invert section. The side-wall sections were poured by means of buckets of 2-cu. yd. capacity hoisted into place by bridge cranes running on the form frames and dumping into small chutes leading to ports through the sides of the forms. In the arch section, the concrete was placed pneumatically by means of concrete guns feeding into 8-in. steel pipes.

Aggregate production comprised the largest haulage problem encountered. The gravel pits designated by the Government were situated on the Arizona side, 11 miles upstream by any feasible route from the dam site, and the topography of the country made it necessary that the raw aggregate from these pits be transported across the river to the gravel screening plant.

A railroad was preferred to an aerial tramway because of its lower cost for equivalent capacity and because the topography is better adapted to railroad operation. Also the contract specified that cement would arrive in bulk in box cars at Boulder City, which assumed the construction of a railroad to the river level. Elaborate studies of steam and electric operation, and a combination of the two, indicated that there was little choice from the standpoint of cost over a seven-year period. The high initial investment in plant and equipment necessary for electrical operation influenced the decision in favor of steam.

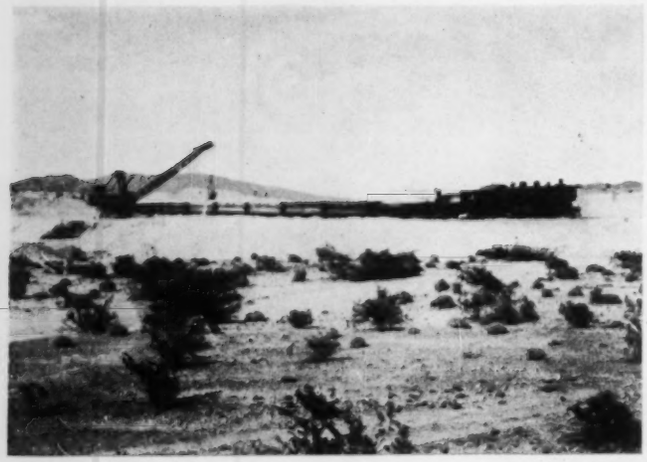
By far the greater part of the material to be transported (27,000,000 out of 32,000,000 tons) is for aggregate. To move this material, Six Companies Inc. have constructed 27 miles of railroad, including storage tracks and sidings. The maximum grade against loaded traffic is 1.75 per cent compensated.

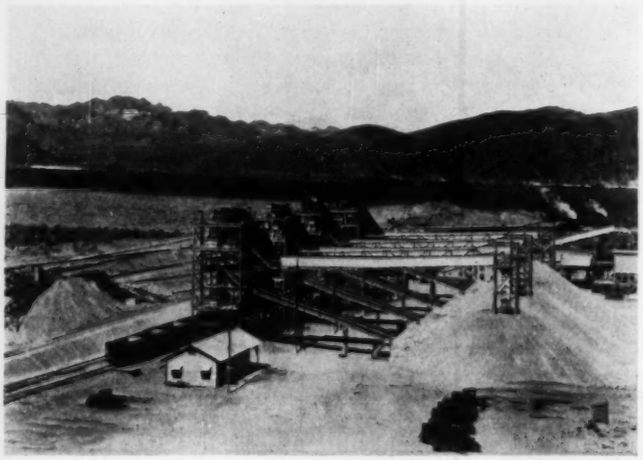
The cost (including the canyon section, which has two tunnels, each over 1,000 ft. long, a 3-span steel bridge 260 ft. long, and almost a half mile of bench cut from solid cliffs) is estimated at over a million dollars, and the total operating costs at over two million.

Gravel is loaded from the pits by an electrically operated dragline. It is interesting to note that trains cross the river from the gravel pit on the Arizona side to the screening plant on the Nevada side on a 1,000-ft. pile trestle bridge. It was considered more economical to construct such a bridge, which might possibly be destroyed by flood water four times during the life of the project, than to install a fixed-type conveyor system on a steel suspension bridge with dump hoppers on each end. The method also allows greater flexibility of operation in the gravel pit, with minimum handling.

When the gravel screening plant was erected, about midway between the gravel pit and the dam site, it was not known what capacity it would be required to main-

tain, whether the gravel would be screened dry or washed, or what the requirements would be for live storage of material at the plant. Hence flexibility appeared to be the most outstanding requirement. The plant was designed, and is now producing, on the basis of 500 tons of finished aggregate per hour, with provision for doubling this capacity. Live storage of aggregate is maintained on such a basis as will permit of almost unlimited increase without interfering with plant operation. The material may be produced either wet or dry, and this feature is changeable at will. Over 9,000,000 tons of raw aggregate will pass through the plant before the dam is completed.





OBTAINING, WASHING, AND MIXING AGGREGATE

Above: Electric dragline excavator with 5-cu. yd. bucket, loads direct into 30-cu. yd. side dump cars; 125-ton locomotives haul 10-car trains to screening plant. Center: Gravel screening plant has capacity of 500 tons per hour and produces sand and four sizes of coarse gravel. Below: the low-level concrete mixing plant. View from the Arizona side, looking down Black Canyon, showing intakes of the Nevada diversion tunnels.

The gravel plant is located approximately 400 ft. above the river level, or just beyond the temporary high-water mark to be made when the diversion tunnels are closed and water is stored to begin power generation, when the dam is half completed. It is a three-way junction in the railroad line, one line dropping into the canyon, a second leading to the gravel pit, and the third going to the canyon rim and Boulder City. This location is about equidistant from the two mixing plants, one in the canyon, the other (to be erected later) at the top of the canyon.

TWO MONSTER CONCRETE PLANTS

The placing of 4,500,000 cu. yd. of concrete with the speed and economy required by present-day construction practice is alone a tremendous task. The problem is further complicated by severe weather conditions. It was therefore necessary for the Six Companies Inc., to design and install a mixing plant that not only would exceed all similar plants in size and capacity, but also would have the most advanced features for controlling consistency and operation, as well as the ability to produce continuously and dependably a concrete meeting the required specifications.

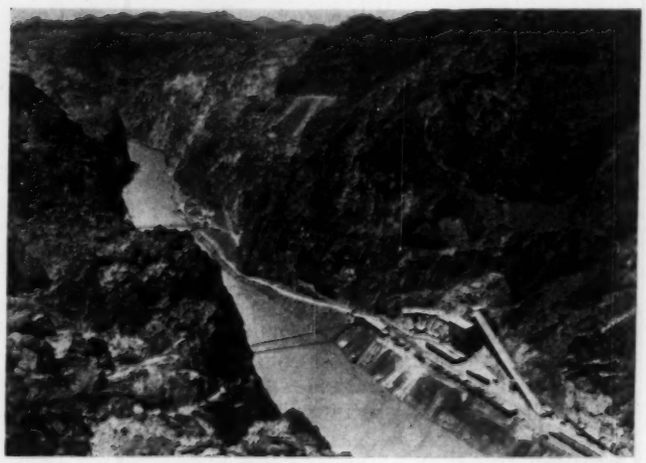
The mixing plant in the canyon is of steel frame construction with laminated timber sides and partitions. At present it contains four tilting mixers of 4-cu. yd. capacity, and has provision for two more. It is estimated that when completed it will produce 170,000 cu. yd. of concrete per month, operating 16 hours daily for 26 working days. Aggregate storage is provided for 14 hours of operation, and cement bins for $3^1/_2$ days.

To meet the specifications, a number of features for weighing, batching, and recording have been devised, considerably in advance of any designs heretofore proposed. All the ingredients of the concrete are weighed by two duplicate sets of weighing batchers, each set feeding two mixers alternately. These include one batcher each for sand and water, one for each of the four sizes of coarse aggregate, and two for cement.

CONCENTRATED MANUAL CONTROL

The discharge from the sand and gravel batchers is conveyed to the mixer hopper by means of a collector belt, and is regulated through a sequence timer to provide ribbon feeding. The cobble and cement batchers discharge directly into the mixer hopper. Water is introduced to the mixer by gravity.

Equipment is provided to record visibly and graphically the time of measurement and the actual quantity of cement, water, and each of the aggregates for each batch. It is readily adjustable to compensate for the varying amounts of moisture contained in the aggregates. The recorder also indicates the relative consistency of each batch.



Operation of each pair of mixers and the batchers for them is entirely controlled from a main operating stand, located above and between the two mixers. Dials for the two cement batchers, and for the water and cobble batchers, are directly in front of this stand. Signal lights show the operator which batchers are in service and when they are filled and ready for discharge. In normal operation, the aggregate batchers are entirely

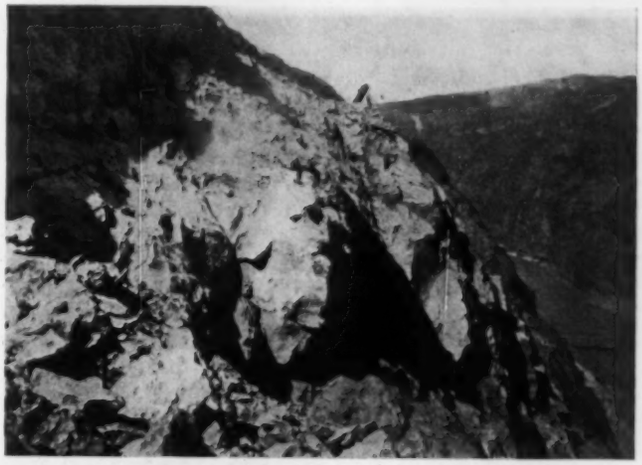
automatic, so that the operator has only to push the discharge button whenever he desires to fill the mixer hopper with aggregate.

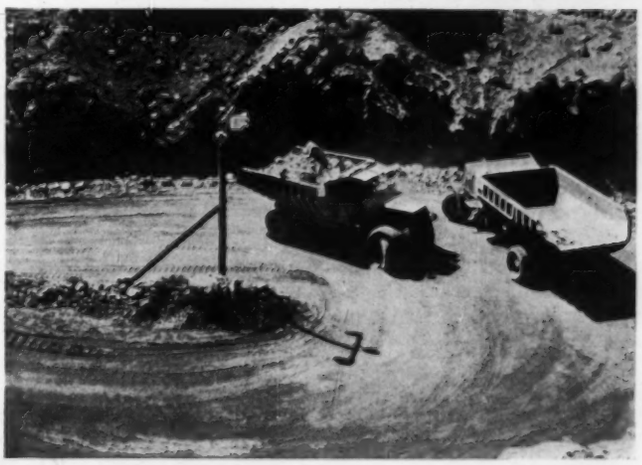
Discharge of the cement batcher is controlled by an air cylinder which has its operating valve at the main control station. Discharge of the water is controlled by a hand-operated valve directly in front of the operator. The entire control station is so arranged that one operator has control of all batching, mixing, and mixer discharge for two mixers. Any batcher may be cut in or out as desired and may be filled by hand control when necessary.

PLANT SCHEDULE FOR DAM CONSTRUCTION

In concreting the foundation of the dam, it is planned to use both motor truck and railroad, as well as a system of traveling cableways above the canyon rim. In excavating the foundations, a large part of the material will be hauled by truck directly from the point of excavation to the dumps. The remainder will go by truck to a large dock, where the trucks will dump into 30-yd. air-dump cars to be hauled by rail about three miles to spoil grounds outside the canyon. It is planned to load the concrete for the lower part of the dam into 8-yd. bottom-dump buckets, which will be placed on standard-gage flat cars and hauled from the mixing plant to the dam site. The concrete bucket will be picked up from the railroad car and transported in one movement by cableway to the point of final placement. For the pouring of the upper part of the dam, 8-yd. buckets will be transported to the loading point of the cableway by trucks.

A system of cableways having a 20-ton capacity was chosen as the logical plant for pouring the concrete in the dam, intake towers, and power houses and that in a part of the spillways. Cableways were considered the most flexible installation; their total cost was lower; and their performance under given conditions was a known factor.





ROAD WORK AND ROCK EXCAVATION

Above: Workmen with pneumatic drills cling to the Nevada abutment of the dam in the construction of the road that will eventually cross on top of the dam. Boom of electric shovel is visible. Center: Two 14-cu. yd. rock trucks passing on a hairpin turn on an 18 per cent grade 800 ft. above the river. Note the road-lighting facilities. Below: Beginning of the 400,000-cu. yd. excavation for the Nevada spillway. A similar spillway will be built on the Arizona side.

The specifications provided that the concrete must be deposited in its final place, thus eliminating the conventional methods of continuous pouring, such as chutes and conveyors.

The system of cableways as designed by Six Companies Inc. is a unique installation in construction practice. At least eight cableways will be required, two of which will have a span of almost half a mile. The total installation will cost nearly $850,000. Because of the extremely irregular topography, a third of this sum will be expended in the excavation and construction of the runways. The cableways over the dam proper will be supported at both ends by traveling structural steel towers ranging in height from 45 to 90 ft. and having a rail speed of 150 ft. per minute. Each track cable will be 3 in. in diameter, of lock wire construction and special steel, and will have an ultimate strength of 550 tons. The cableways are designed to handle a specially constructed 8-yd. concrete bucket at a hoisting speed of 300 ft. per minute and a conveying speed of 1,200 ft. per minute.

ADDITIONAL CONSTRUCTION ITEMS

In all, the contract calls for 119 separate bid items. Some of the larger ones not previously mentioned in this article include about 300,000 cu. yd. of rock excavation, carved from the canyon walls for the bases of intake towers; over 1,000,000 cu. yd. of earth fill for cofferdams; an estimated amount of 150,000 cu. yd. of loose rock stripped from the precipitous walls of the canyon; the placing of 35,000,000 lb. of reinforcement bars and rails; installation of 6,600,000 lb. of pipe, fittings, and valves in the dam; and the transportation from mill to canyon of approximately 15,000,000 lb. of pressure penstocks, some of them 30 ft. in diameter and 3 in. thick. An additional feature is the expenditure of over $1,000,000 for the finest construction camp ever built, at Boulder City, Nev.

Uplift Pressure in Masonry Dams

Recorded Measurements Summarized and a New Theory Developed

MOVEMENT and pressure of water under dams were among the subjects selected for investigation by the Society's Research Committee on Irrigation Hydraulics. At the request of the committee, Mr. Houk assembled available data on measurements of uplift pressure in a paper which was presented before the joint session of the Irrigation and Power Divisions on July 7, at the Annual Convention in Yellowstone Park. In this paper, here published practically in its entirety, Mr. Houk points out that the curves of observed pressures are rather closely grouped and have values usually less than those represented by a straight-line variation in pressure intensity between two-thirds of full pressure at the heel of a dam and zero at the toe.

Before the same joint session at Yellowstone Park, Mr. Henny reviewed the effect of uplift pressure on the design of gravity dams and presented an original theoretical treatise based on the available measurements of pressure and on other research. The committee believes this analysis will be far reaching in its effect and hopes to publish it in full at some future date. A brief abstract of Mr. Henny's paper is here made immediately available to readers.

Measurements at Existing Structures

By IVAN E. HOUK

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SENIOR ENGINEER, U.S. BUREAU OF RECLAMATION, DENVER, COLO.

UP TO the present time, measurements of uplift pressure have been made on only ten masonry dams built on rock foundations. In seven instances the measurements were taken only at the contact surfaces between the foundation rock and the masonry, and in three instances they were made at construction joints within the body of the dam as well as at the foundation level. Of these ten dams, here listed, the last three are those on which observations were made at construction joints as well as at the base:

Oester, Germany	Pit River No. 3, Calif.
Neye, Germany	American Falls, Idaho
Willwood, Wyo.	Bull Run, Ore.
Medina, Tex.	Gibson, Mont.
Brule River, Wis.	Ariel, Wash.

The Willwood, Medina, Brule River, and American Falls dams are straight gravity structures having maximum heights of 70, 166, 70, and 80 ft., respectively. The Oester, Neye, Pit River No. 3, and Bull Run dams are curved gravity structures with maximum heights of 131, 100, 112, and 200 ft., respectively. The Gibson and Ariel dams are arched structures having maximum heights of 200 and 313 ft., respectively. All the foundations were grouted, except at the Brule River and Oester dams, and all of them were drained except at the Oester, Neye, and Willwood dams. Drains were installed under the apron at the Willwood Dam, but not under the main part of the structure.

DATA NEEDED FOR DESIGN

Pipes for taking measurements of uplift pressure have been installed at the foundation level and at various horizontal joints in the Owyhee Dam, a 405-ft. curved gravity dam now being completed in eastern Oregon. Plans for similar installations have been prepared for the Madden Dam, a 210-ft. straight gravity dam now under contract in the Panama Canal Zone. Possibly investigations of uplift pressure have been made at other existing dams; however, no descriptions of such measurements have been found in engineering literature.

In order to make proper allowances for the effects of uplift pressure in designing masonry dams, the engineer needs to know two things; first, the intensity of this pressure at different locations between the upstream and downstream faces of the dam; and second, the proportion of the horizontal cross-sectional area on which this pressure acts. Thus far, experimental investigations of uplift, either in the laboratory or at existing dams, have supplied no satisfactory data bearing on the second question. In the case of porous materials, some valuable information has been secured, and this may be applicable to masonry dams on coarse sandstone

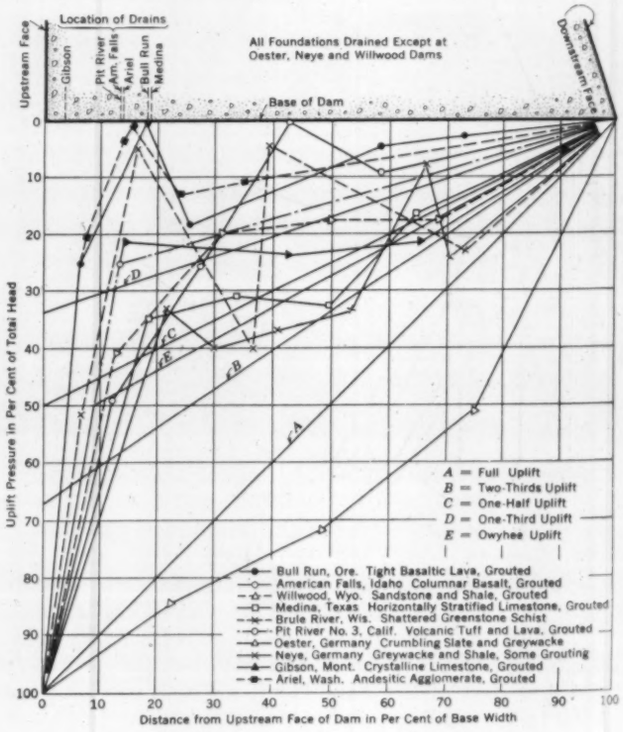

Location of Drains
Upstream Face
Gibson
Pit River
Am. Falls
Ariel
Bull Run
Medina
All Foundations Drained Except at Oester, Neye and Willwood Dams
Downstream Face
Base of Dam
Uplift Pressure in Per Cent of Total Head
Distance from Upstream Face of Dam in Per Cent of Base Width
A = Full Uplift
B = Two-Thirds Uplift
C = One-Half Uplift
D = One-Third Uplift
E = Owyhee Uplift
Bull Run, Ore. Tight Basaltic Lava, Grouted
American Falls, Idaho Columnar Basalt, Grouted
Willwood, Wyo. Sandstone and Shale, Grouted
Medina, Texas Horizontally Stratified Limestone, Grouted
Brule River, Wis. Shattered Greenstone Schist
Pit River No. 3, Calif. Volcanic Tuff and Lava, Grouted
Oester, Germany Crumbling Slate and Greywacke
Neye, Germany Greywacke and Shale, Some Grouting
Gibson, Mont. Crystalline Limestone, Grouted
Ariel, Wash. Andesitic Agglomerate, Grouted

FIG. 1. MAXIMUM OBSERVED UPLIFT PRESSURE Under Existing Gravity Dams on Rock Foundations

foundations; but no adequate data have been collected for horizontal construction joints within the concrete, or for contact areas between the base of a dam and its actual rock foundations.

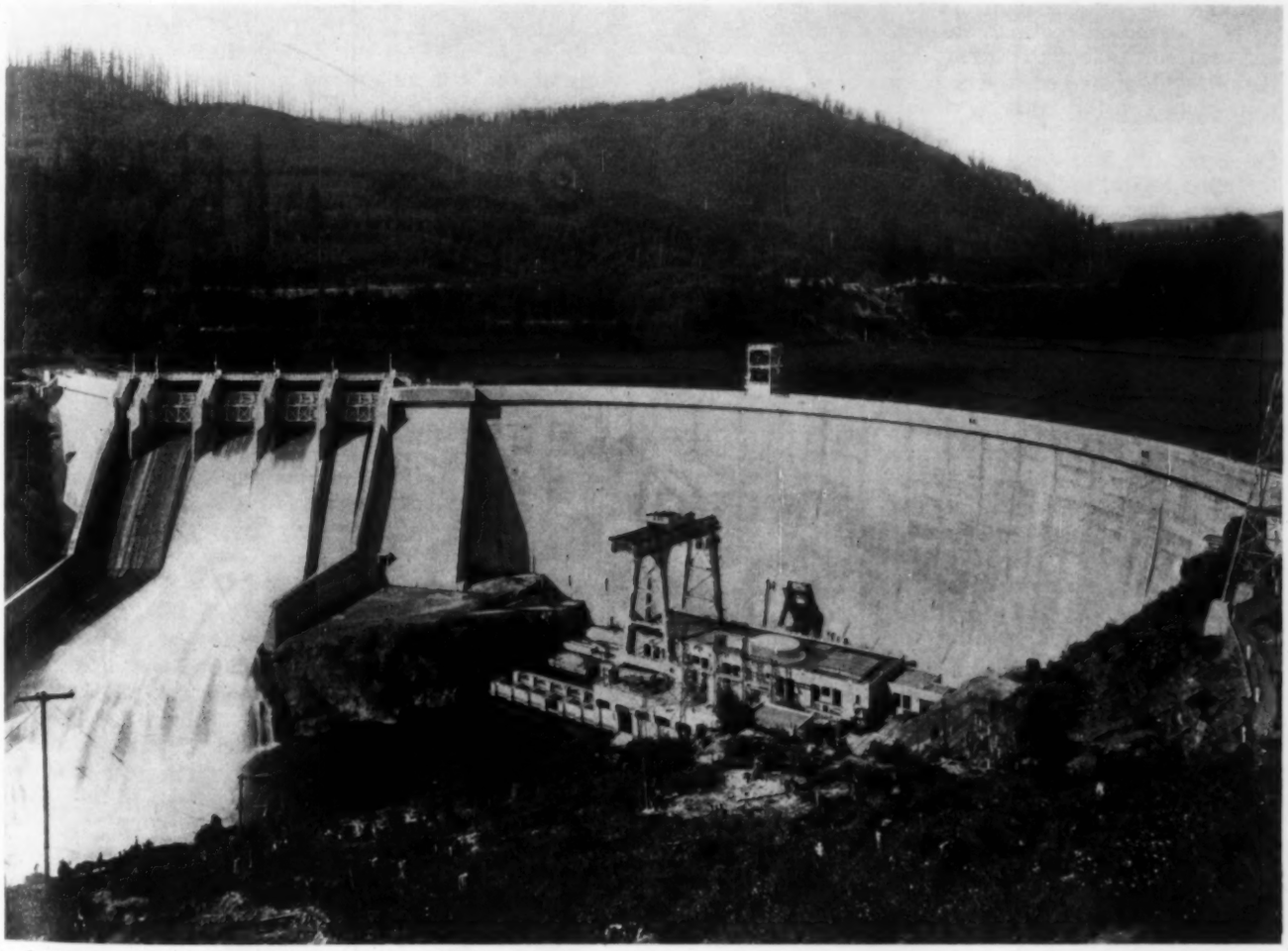
Measurements of uplift pressure at masonry dams have furnished fairly satisfactory data, although the results of the measurements are naturally somewhat erratic. The observed pressures vary widely at different dams, at different locations in the same dam, and with different elevations of reservoir surface. However, when the maximum observed pressures are reduced to a common basis, they lead to fairly definite conclusions.

MAXIMUM OBSERVED PRESSURES

The accompanying diagram, Fig. 1, shows the maximum uplift pressures observed at the foundations of the ten dams previously listed. In most cases there were several additional measuring pipes, which showed smaller uplift pressures. Measurements during reservoir stages at, or very close to, the ultimate heights to be expected were available for practically all the dams represented. Observed pressures have been corrected to tailwater level where the base of the dam is below tailwater, and the results are shown as percentages of the total head. This was taken as the height of the reservoir water surface above the base of the dam in cases where the foundation was above tailwater; and as the height of the reservoir water surface above tailwater in cases where the base of the dam was below tailwater.

Locations of the uplift measurements with respect to the upstream faces of the dams are shown in Fig. 1 as percentages of the total base width, and the locations of the foundation drains are also indicated. Curves for uplift pressure have been arbitrarily extended as straight lines from the values observed at the pipes farthest upstream to the value of 100 per cent at the upstream edge of the base. Similarly, straight lines have been drawn from the values observed at the pipes farthest downstream to the 0 per cent value at the downstream edge. The latter extension is probably an accurate assumption; the former may indicate uplift pressures slightly greater than those actually existing.

For comparative purposes, the curves of intensity of uplift pressure usually assumed in designing concrete dams are included in Fig. 1. Curve *A* is a straight line drawn from full reservoir pressure at the upstream face of the dam to zero pressure at the downstream face. Curve *B* shows the uplift pressures which, if applied to the full horizontal area, will give the same effect as the Curve *A* pressures applied to two-thirds the area. Similarly, Curves *C* and *D* indicate the uplift pressures which, if applied to the full horizontal area, will give the same effects as the Curve *A* pressures applied to one-half and one-third the areas, respectively. Curve *C* represents the assumption used in designing the Willwood and Gibson dams; and Curve *D* indicates that employed in the design of the American Falls Dam. Curve *E* shows the assumption for the Owyhee Dam, the uplift pressure in this case being reduced to one-half the reservoir pressure at the location of the drains, and the pres-



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ARIEL DAM AND POWER HOUSE, LEWIS RIVER, WASH.

Elaborate Provisions Have Been Made to Measure Uplift Pressures of This Structure

sures represented by the curve being applied to the full horizontal areas. It is understood that a curve similar to Curve *E*, drawn to one-fourth the reservoir pressure at the location of the drains, instead of one-half, was used in designing the Bull Run and Ariel dams. Data regarding the assumptions used in the design of the other dams represented on the diagram are not at hand.

ASSUMPTIONS FOR DESIGN

Since only the maximum observed pressures are shown on the diagram, it is evident that the design assumptions were adequate in the case of the American Falls, Willwood, Gibson, Bull Run, and Ariel dams, the only ones for which data on design assumptions are available.

It is interesting to note that the Bull Run and Ariel dams, for which the assumed uplift conditions were least severe, are more nearly free from uplift pressure than any of the other structures represented. It is understood that unusual care was exercised in the excavation and preparation of the foundation for the Ariel Dam, and that the foundation rock at the Bull Run site was unusually water-tight in its original state and that it was thoroughly grouted and drained during the construction of the dam.

Leaving out of consideration the Oester Dam in Germany, where the foundation rock was not ideal and where, apparently, modern precautions of grouting and draining the foundation were not taken, it will be noticed that the observed pressure curves are fairly well grouped and are located mostly between Curve *B* and the base of the dam. This would indicate that, if the foundation rock is satisfactory and the grouting and draining adequate, uplift assumptions for design should seldom exceed the application of Curve *A* to more than two-thirds the horizontal area of the base.

Curve *E*, or a modification of it, such as was used for the Bull Run and Ariel dams, seems to be at present the most logical uplift assumption to make in designing masonry dams on rock foundations. In selecting such a curve, the proportion of reservoir pressure assumed at the location of the drains should be varied in accordance with the character of the rock foundation. However, it seldom if ever will be necessary to place the value at more than one-half the reservoir pressure.

Curve *E* is applied to the full horizontal area. This is undoubtedly the best procedure to follow at the present time. Some time in the future definite data may be secured regarding the ratio of the effective area on which uplift pressures act to the total horizontal area, for different kinds of foundation rock. When that time comes it may be possible to apply the uplift pressure intensities represented by Curve *E* to some part of the horizontal area instead of to all of it.

UPLIFT AT CONSTRUCTION JOINTS

Thus far no measurements of uplift pressure at horizontal construction joints have been published. No appreciable uplift pressures have been observed at construction joints in the Gibson Dam, even under heads as high as 112 ft., and at distances from the upstream face of the dam as small as 6 ft. It is understood that no appreciable pressures have been noted at construction joints in the Ariel Dam under heads up to 115 ft.; and that only comparatively small pressures have been found at the Bull Run Dam, under heads as high as 166 ft., and at distances from the upstream face as small as 5 ft. Consequently, for purposes of design, it seems safe to conclude that assumptions of uplift pressure applicable to the foundation level of a masonry dam will be more than ample for horizontal construction joints at different elevations above the base.

ACKNOWLEDGMENTS

The technical studies which formed the basis for this article were made in the Denver office of the Bureau of Reclamation. All the design studies and investigations of the bureau are under the direction of J. L. Savage, M. Am. Soc. C.E., Chief Designing Engineer; all engineering and construction work is under the general direction of R. F. Walter, M. Am. Soc. C.E., Chief Engineer, whose headquarters are in Denver, Colo.; and all the activities of the bureau are under the general charge of Elwood Mead, M. Am. Soc. C.E., Commissioner of Reclamation, whose headquarters are in Washington, D.C.

Effect of Uplift on Stability of Straight Gravity Dams

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UNTIL the beginning of this century, the middle-third method was commonly used to find the safe dimensions of a gravity dam. It merely assumed the absence of tension in the masonry, near the upstream face with reservoir full, and near the downstream face with reservoir empty. The failure of the Austin (Texas) Dam in 1900 called attention forcibly to the danger of sliding aided by percolation, in reducing the friction coefficient between foundation layers by the presence of water.

UPLIFT PRESSURE

Experimental data on ten dams are presented in Mr. Houk's article. One of these dams (in Oester, Germany) stands on an unusually bad foundation and is in a class by itself. The pressure diagrams show that the measured uplift pressures fall well below a straight-line gradient with full water pressure at the heel (upstream), and no pressure at the toe (downstream). The results of this

TABLE I. AVERAGE UPLIFT PRESSURE MEASUREMENTS

DAMS	DRAINED	UNDRAINED
Neye, Germany	...	0.349 *H*
Brule River, Wis.	0.336 *H*	...
Medina, Tex.	0.290 *H*	...
Gibson, Mont.	0.248 *H*	...
Willwood, Wyo.	...	0.246 *H*
Pit River, Calif.	0.190 *H*	...
American Falls, Idaho	0.153 *H*	...
Ariel, Wash.	0.121 *H*	...
Bull Run, Ore.	0.097 *H*	...

research may be expressed in terms of average uplift pressure under the entire base in units of height of water, H, as given in Table I.

Drainage is important. In view, however, of possible future neglect to keep drains clean, no dependence should be placed on them for safety. It is proposed to use an even gradient of pressure from H at the heel to 0 at the toe in estimating uplift. This is equivalent to an average of 0.5 H, which is seen to be from 1.5 to 5 times the pressure measured in drained dams, and from 1.5 to 2 times that found in undrained dams.

The uplift force with which the designer of dams is principally concerned is the product of unit pressure and area. Area is much more difficult to measure and yet it may be more important than unit pressure. The pore volume of concrete in dams should probably be taken at from 10 to 12 per cent. The minimum uplift area may therefore likewise be placed at 12 per cent of the gross area.

Experiments made by S. H. Woodard, M. Am. Soc. C.E., proving that pore water can exert stresses, and other experiments made at Portland, Ore., and Pasadena, Calif., show that it is easy to unite concrete surfaces so as to practically prevent water movement along the joint. This conclusion is confirmed by the fact that uplift pressure was not found at concrete construction joints in the great majority of research tests, covering three of the ten dams previously listed. On the other hand, uplift pressure was noted at nearly all points located at the junction of concrete and rock. It is apparent that water movement is more free at the base than through the concrete, which fact may well be due to the difference in characteristics of the two materials.

It seems probable that, where unit loading exceeds full reservoir head, the uplift pressure area, even at the base of a dam, will be held down to pore area with an allowance for lack of homogeneity. It may follow likewise that where the loading is lighter, the uplift pressure area will be greater. As no definite information is available, all that the engineer can do until research reveals further facts, is to follow his best judgment.

Many small dams have been built with a water loading at the toe for full reservoir of less than four-tenths of the full water head, which practically agrees with a downstream slope of 0.7. As in general no progressive increase of percolation or uplift pressure was noted, it may be argued that the effective uplift area at the water face did not exceed 40 per cent. This percentage equals over three times the pore area and is probably a fair and safe estimate under such loading conditions, careful field work being taken for granted.

For lighter loading it may be supposed that this percentage increases rapidly. On the other hand, with heavier loading, the potential percentage will probably decrease and the somewhat arbitrary relation of Table II may be assumed on the premise that the dam is founded on good rock.

It would be logical, when considering the entire base, to use the percentages in the last column of the table. The condition at the water face is likely to have a predominating influence on percolation. It must be remembered, moreover, that it is imperfection of field work and rock base that governs these percentages and that an accurate appraisal of this imperfection is practically impossible. The more conservative course would be to use for the entire base the percentages listed in the second column of the table.

CONCLUSIONS SUMMARIZED

Uplift Pressure—*Uplift pressure can be estimated with reasonable accuracy. It is ordinarily less than 50 per cent of full reservoir head at 10 per cent of the base length from the water face and diminishes to zero at the toe. The assumption that pressure decreases uniformly from full head at the heel to zero at the toe is abundantly safe.*

Uplift Area—*The proportion of effective uplift area to the total area of any given horizontal section is not known. The area along a concrete construction joint approaches the pore area in well built work. The effective uplift area between concrete and foundation rock is probably larger unless unit loading at the heel is close to full water pressure. The potential relative pressure area is likely to decrease as vertical loading increases.*

Sliding Factor—*As commonly used, the sliding factor for judging the safety of a dam against downstream movement is not a logical criterion except in the case of extended horizontal stratification planes in the foundation rock. When so used a suitable factor of safety should be introduced.*

Shear-Friction Factor—*This is a more logical guide than the sliding factor and is to be preferred even though foundation shear resistance cannot be accurately estimated. Its use brings out the effect of height on the safety of the dam and emphasizes the value of construction methods which will fully develop shear resistance.*

Levy Requirement—*The Levy requirement that the masonry loading at the water face must not be less than the water pressure at that elevation is in ordinary cases excessive for dams lower than 500 ft.*

UPLIFT FORCE

Uplift pressure should be estimated on the basis of full water head at the water face, diminishing evenly to zero at the toe. This estimate may be combined with the suggested pressure areas in order to arrive at the uplift force.

In the study of the stability of dams, it is convenient in illustrating principles, to consider a straight dam of infinite length, with triangular cross section, having a horizontal base, L, and a vertical water face, H, the water being assumed to stand at the apex of the triangle. Table III shows the uplift force at the base for various values of L/H, straight-line stress distribution being assumed as well as a specific gravity of 2.4 for wet concrete.

For dams with downstream slopes steeper than 0.707 the uplift force at the water face may exceed the vertical loading. Such a condition is to be carefully avoided as it may result in slow progressive failure in tension. This in turn may be followed by an increase in the uplift area and a shift of loading toward the

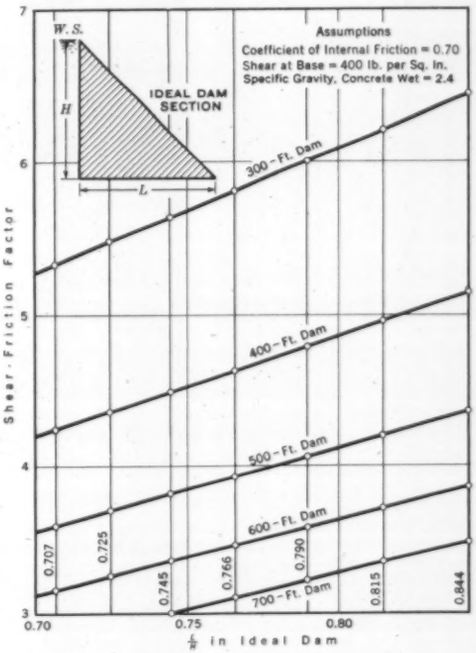


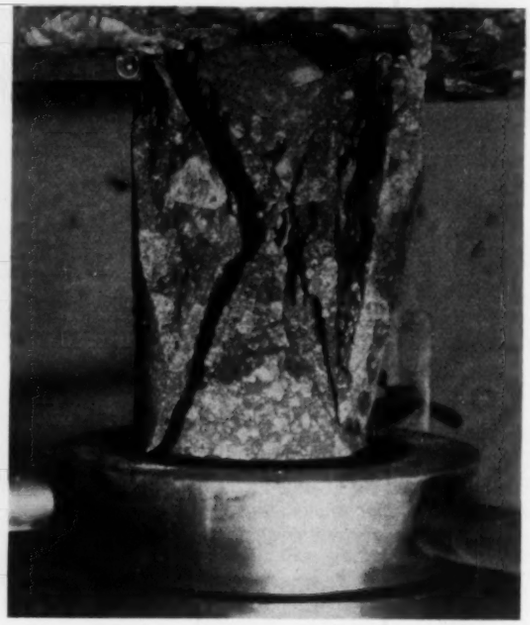
FIG. 1. RELATION OF SHEAR-FRICTION FACTOR TO $\frac{L}{H}$ IN IDEAL DAM

toe. The consequences of such a shift have been studied and will be referred to later.

SLIDING FACTOR ANALYZED

The sliding factor, F_s, commonly used for judging the safety of a dam, may be expressed for the ideal triangular dam section 1 ft. long as follows: $F_s = \frac{P}{W - U}$ in which P is the horizontal water pressure against the face of the dam, W the weight of the concrete, and U the uplift force. It is usually held that if F_s falls below some selected value for the coefficient of friction, such as 0.75 or 0.70, the safety against sliding is satisfactory.

Various objections may be raised to the use of such a factor for judging safety. It does not in any way indicate the real factor of safety, which by its use would, in most cases, appear to be close to one. It is the same for all heights of dam having a similar section. If applied to downward inclined planes through a dam it would indicate certainty of sliding along such planes. By its use a dam on horizontally stratified rock would be adjudged as safe as one on rock with or without inclined stratification.



COMPRESSION TEST ON ROCK CORE FROM HOOVER DAM ABUTMENT
Angle of Shear Break 65 to 70 Deg.

SHEAR-FRICTION FACTOR DEVELOPED

It is therefore proposed to substitute for the sliding factor a different expression, which will recognize the one important element heretofore ignored, namely shear. The expression is as follows: $Qf = (W - U)f + Ls$, in which s is the unit shear resistance, and f is the coefficient of internal friction.

Shear breaks in cylinders tested in compression appear to demonstrate cooperation between shear resistance and internal friction, both before and at the time of failure. This cooperation may also be called increased shear resistance due to compression normal to the plane of shear. The usual angle of break shows that concrete with a strength of 3,000 lb. per sq. in. in compression, has a shearing strength of about 800 lb. per sq. in. in combination with an internal friction coefficient of 0.70. At least the phenomenon can be logically explained in this manner, although it is admitted that there is as yet no complete proof that the relation between loading normal to the plane of shear and increase of shear resistance is a constant, as has been assumed here. Research on this subject is now under way and may in time furnish definite information.

It will be seen that the shear-friction factor, Q, is in fact the factor of safety against downstream movement. Since P, W, and U are all proportional to H^2, and L is proportional to H, it follows that the equation may be given the following general form: $Q = mf + \frac{n}{H}S$. The first part of the equation is expressive of internal friction and is independent of H, while the second part is added as a result of shear. The value of Q is seen to become smaller as H increases, or, in other words, the liability of downstream movement is greater for a high dam than for a low one of the same proportions. This relationship is shown in Fig. 1.

The shear-friction factor, besides being more logical, brings out more clearly the importance of developing, by proper construction methods, the shear resistance at all concrete construction joints, and particularly at the junction between the concrete and the rock. Its use also emphasizes the comparatively low factor of safety of a dam on a horizontally stratified foundation, on or within which sliding may take place without overcoming shear resistance.

An unusually seamy rock base, within which it is assumed that rather free water movement from below cannot be prevented and on which excessive effective uplift pressure area must be expected, presents an interesting case. There is a possibility that serious shifting of load will occur as well as penetration of water under full head to a point on the base where the loading will be equal to full water height. On the basis that foundation shear can exist only for the remaining part of the base, values of L/H can be determined that are amply safe for various heights of dam. Such values for dams lower than 500 ft. are still well below the limit recommended by Levy.

The suggested method of ascertaining the safety of a dam against downstream movement was applied to certain existing structures to illustrate the superior logic and usefulness of the shear-friction factor.

CONVENTION PAPERS PUBLISHED

All papers from the Yellowstone Convention, with the exception of two, are included in condensed form in this issue of CIVIL ENGINEERING. *The paper on "Forests and Stream Flow," by W. G. Hoyt, M. Am. Soc. C.E., and H. C. Troxell, Assoc. M. Am. Soc. C.E., presented at the general technical meeting on July 6, has already appeared in* PROCEEDINGS *for August 1932. That by H. H. Hatch, M. Am. Soc. C.E., entitled "Tests for Hydraulic Fill Dams," read before the joint session of the Irrigation and Power Divisions on July 7, 1932, is expected to appear in a later number of* PROCEEDINGS.

TABLE II. ASSUMPTIONS REGARDING UPLIFT PRESSURE UNDER DAMS

Vertical Loading at Upstream Toe	Effective Uplift Pressure Area		
	At the Heel	At the Toe	Average
0.4 H	3 x (pore area), 40%	25%	32½%
0.6 H	35%	25%	30 %
0.8 H	30%	25%	27½%
1.0 H	2 x (pore area), 25%	25%	25 %

TABLE III. UPLIFT UNDER TRIANGULAR-SHAPED GRAVITY DAMS

$\frac{L}{H}$	Heel Loading	Relative Area Subject to Uplift	Total Uplift Force
0.707	0.4 H	0.40	0.20 HL = 0.141 H^2
0.725	0.5 H	0.375	0.188 HL = 0.136 H^2
0.745	0.6 H	0.35	0.175 HL = 0.130 H^2
0.766	0.7 H	0.325	0.163 HL = 0.124 H^2
0.790	0.8 H	0.30	0.150 HL = 0.118 H^2
0.815	0.9 H	0.275	0.138 HL = 0.112 H^2
0.844	1.0 H	0.25	0.125 HL = 0.106 H^2

HINTS THAT HELP

Today's Expedient—Tomorrow's Rule

The minutiae of everyday experience comprise a store of knowledge upon which we depend for growth as individuals and as a profession. This department, designed to contain practical or ingenious suggestions from young and old alike, should afford general pleasure not unmixed with profit.

Calculating Stiffness of Corrugated Steel Sheets

By ROBINS FLEMING

AMERICAN BRIDGE COMPANY, NEW YORK

VERY little has been written on the strength, and almost nothing on the stiffness, of corrugated metal. Rankine's formula for the strength of corrugated metal sheets has been used almost exclusively from the time of its publication in the first edition of his *Civil Engineering*, 1862, until the present. For a breaking load, W, in pounds, uniformly distributed on a sheet supported at the ends, it is:

$$W = \frac{\frac{32}{15} f b h t}{l}$$

in which f is the modulus of rupture; b, the breadth of the sheet; h, the depth of the corrugation from ridge to furrow; t, the thickness of the metal; and l, the span. All dimensions are in inches.

After diligent search for any formula that could be used for the stiffness of corrugated metal, I finally found an article, "Strength and Stiffness of Corrugated Iron—Remarks on the Experiments of Mr. J. H. E. Hart, C.E.," in *The Engineer* for November 13, 1868, Vol. 26, page 355. This was signed "W. J. M. R., Glasgow University, 2nd November 1868." The author is no other than the eminent Professor Rankine. The experiments made by Mr. Hart are recorded in *The Engineer* for October 16, 1868, reprinted from the *Bombay Builder* for August of the same year. The results of Mr. Hart's experiments agreed fairly well with the Rankine formula for strength.

The stiffness of a sheet acting as a beam is indicated by the load that it can carry with a given deflection. It is the reciprocal of the deflection—that is, if d is the maximum deflection of a beam of span l, $\frac{d}{l}$ is a measure of its stiffness.

The last paragraph of the article by Professor Rankine reads as follows:

"To find the probable deflection (v_1), under half the breaking load, of a given sheet of corrugated iron; l being the span.

$$v_1 = \frac{n''}{m'} \times \frac{f}{2E} \times \frac{l^2}{4h} \quad \ldots\ldots\ldots\ldots [11]$$

in which $\frac{f}{2E} = 0.00096 = \frac{1}{1{,}040}$; and $\frac{n''}{m'}$ has the following values: sheet loaded in middle of span, $^2/_3$; sheet uniformily loaded, $^5/_6$."

In the foregoing, f, the modulus of rupture, is assumed to be 39,100 lb. per sq. in.; E, the modulus of elasticity, is taken at 20,400,000; h is the depth of corrugation from ridge to furrow; and l is the length of span in inches. As t (thickness of metal) is a factor in determining the breaking load, and b (breadth of sheet) is assumed to be unity, they do not enter into Rankine's Equation 11.

For steel, assuming f to be 60,000 lb. per sq. in. and E to be 30,000,000, Rankine's Equation 11 gives a load at the middle of less than one-fourth the breaking load:

Deflection with sheet loaded in middle =

$$\frac{2}{3} \times \frac{60{,}000}{4 \times 30{,}000{,}000} \times \frac{l^2}{h} = 0.000083 \frac{l^2}{h}$$

Deflection with sheet uniformly loaded =

$$\frac{5}{6} \times \frac{60{,}000}{4 \times 30{,}000{,}000} \times \frac{l^2}{h} = 0.000104 \frac{l^2}{h}$$

Thus, the deflection of a sheet 5 ft. long, with corrugations $^5/_8$ in. deep, having a weight in the middle of one-fourth the breaking load, is 0.48 in.; and when the weight is uniformly distributed the deflection is 0.60 in.

The engineer can assume other values for f and E if he desires to do so. He will recognize that the modulus of rupture derived from the flexure formula is approximate. Experimental data regarding the stiffness of corrugated metal are badly needed.

Soundings by Triangulation Intersection

By WILLIAM D. AYERS

ASSOCIATE MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS

MANAGER, ISLIP OFFICE, WALLACE H. HALSEY, INC.

WHAT is believed to be a new method of plotting soundings was recently given a successful trial by Wallace H. Halsey, consulting engineer of Southampton, Long Island, N.Y. During the summer of 1931, Mr. Halsey was commissioned to make an 18$^1/_2$-mile sounding survey in Great South Bay, Long Island. I was in charge of the work, which consisted of establishing major triangulation control, minor controls, levels, and bench marks; locating old division lines, oyster beds, necessary artificial features, and channel markers; and taking soundings over the 18$^1/_2$-mile length. Practically all the soundings were taken by triangulation intersection. As there were a large number of soundings to be plotted, and as the greater part of these ranged from one-half mile to a mile and a half off shore, it was necessary to devise some quick and accurate method of plotting them on the detail maps that were drawn up on a scale of 200 ft. to the inch.

Large paper protractors with a diameter of 14 in. and a graduation of 15 min. were obtained. These were cut in half, giving a protractor with a range of 180 deg. A small piece of kodak film double in thickness was attached to the center part of the protractor. A hole was pricked through the exact center of this so that a pin could be placed through it and through the control point on the detailed map. With a protractor tacked down and oriented on each of two control points, it then became a question of laying off the angles and obtaining the points of the soundings by intersection.

At first, long sliding arms were attached to the protractors for the purpose of laying off the angles, but they

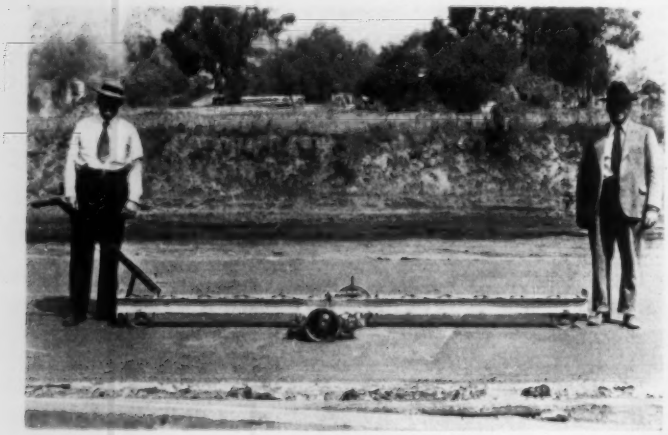
were found slow, clumsy, and inaccurate, besides being too short to plot soundings that were long distances from shore. It was then decided to attach lengths of thread to the pin at the center of each protractor and plot the soundings from the intersection of the two threads, which would be stretched over the desired angle on the protractor. Lengths of silk dental floss were attached to the pin on each of the two protractors. The floss was stretched taut over the angles to be laid off on each protractor, and the intersection of the two was marked by a pencil point. It is important that, at the time of plotting each sounding, the floss be drawn taut in order to obtain the desired degree of accuracy. For this reason, dental floss was used, as it possesses a high degree of tensile strength. With one man calling off the angles from the two notebooks, and a man at each of the two fixed protractors, the soundings were quickly and accurately plotted.

Pavement Roughness Recorded

By CLARAN F. GALLOWAY

ASSOCIATE MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS
ASSISTANT CHIEF SURVEYOR, LOS ANGELES COUNTY ROAD DEPARTMENT, LOS ANGELES, CALIF.

A DEVICE similar to a recording straight-edge, which was recently constructed by the Los Angeles County Road Department for the purpose of measuring the smoothness of pavements, has proved very valuable in helping to produce smoother riding surfaces. This machine consists essentially of a light metal frame, approximately 6 in. high, 6 in. wide, and $10\frac{1}{2}$ ft. long,



RECORDING SURFACE METER

with a metal wheel mounted at each end. The distance between the bearings from center to center is just 10 ft. Exactly in the center of the frame, a wheel is mounted on a vertical slide guided by roller bearings that permit the wheel to rise and fall freely to maintain contact with the road surface when the device is wheeled along the pavement.

The motion of this wheel with respect to the frame is transmitted to a fountain pen which records it on profile paper. The relationship between the movement of the pen and that of the wheel may be fixed at any desired ratio. In the instrument used by the Los Angeles County Road Department, the pen moves two units to each unit of vertical movement of the central wheel. The profile paper is moved across a light metal table, by a train of gears actuated by a rubber-tired driving wheel in contact with the road surface, at such a speed that the graph produced has a horizontal scale of 1 in. equals 10 ft.

An empirical method of rating has been tentatively adopted, which may be briefly described. Deviations are counted and given the following weights:

0 in. to $\frac{1}{8}$ in. = 0
$\frac{1}{8}$ in. to $\frac{1}{4}$ in. = 1
$\frac{1}{4}$ in. to $\frac{3}{8}$ in. = 2
$\frac{3}{8}$ in. to $\frac{1}{2}$ in. = 3

The sum of the weighed deviations is then divided by the product of the number of graphs made of each section of roadway, and the length of the section in miles. It will be seen that a surface with no deviation exceeding $\frac{1}{8}$ in. will have a rating of zero by this method.

Graphs were made of a number of roads constructed prior to the use of the surface meter. The roughness ratings on the pavements were all above 150, and some of them were as high as 500. The roughness ratings on stretches constructed since the surface meter has been in use indicate a steady improvement in the quality of workmanship. The first job on which the surface meter was used during construction has a rating of 90.6. At this time such a rating was considered excellent. Before long, however, a rating above 50 was considered high, and then above 40. The last piece of work constructed for which we have a complete record, has a roughness rating of 35.8.

Until recently, the rating for the first day the meter was operated was likely to be rather high. Then, day by day, it showed a decline. For example, on the first day's construction one job had a rating of 72.8; the following day the rating was 63.6; the third day it dropped to 16.8; and since then has steadily continued to decline, finally achieving a rating of 3.8 on the last

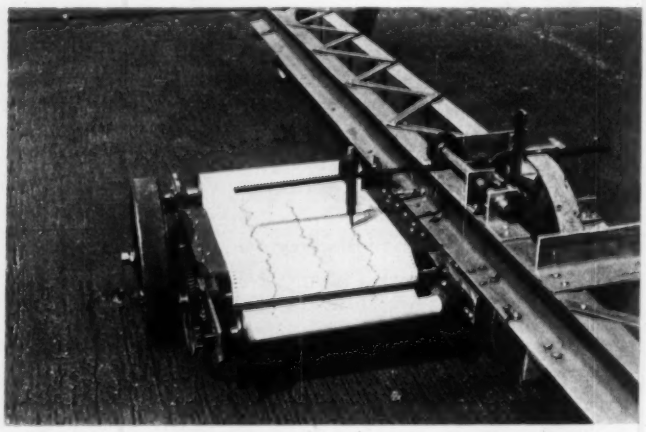


CHART BEING MADE BY RECORDING SURFACE METER
Note That a Number of Graphs May be Made on the Same Chart Without Overlapping

day's run. Now, however, paving crews and resident engineers are becoming more skillful as a result of this continual check and comparison, so that the first day's rating for each job is usually comparatively low.

In addition to the information secured by comparing the roughness ratings of various sections of pavement, a detailed study of high and low places shown on the chart often reveals some recurring or peculiar condition that can be explained by the method of construction. Often this information has made it possible to eliminate the cause of deviation and to effect considerable improvement through changes in technic and equipment.

OUR READERS SAY—

In Comment on Papers, Society Affairs, and Related Professional Interests

D. C. Linsley Had Varied Career

To the Editor: Permit me to add a word of information about D. C. Linsley, a part of whose diary appeared in the June number of Civil Engineering. Knowing that he had been an engineer in Vermont, I gathered some facts about him while on a recent vacation there. Some of this information has already been given by C. W. Linsley in the letter published in the July issue, but I thought that the following additional items, which were obtained from his daughter-in-law, Mrs. J. H. Linsley, and other authentic sources, might be of sufficient interest to warrant publication.

D. C. Linsley was primarily a promoter and builder of railroads. Like other railroad promoters of that day, he did not become permanently rich; some projects were profitable and others were the reverse. He was the first "City Engineer and Surveyor," of Burlington, Vt., 1868–1869. At least, he was the first of whom there is a definite record, and he was mayor of Burlington in 1870. The tunnel referred to by his nephew was, according to the dates cut into its portals, begun Nov. 1, 1860, and completed May 17, 1861, and it is still in regular use. It is from 300 to 400 ft. long and is curved for its entire length, the proximity of the lake to the hill making the curve necessary. It is in the shape of a horseshoe and has a brick lining with limestone portals. Construction of the tunnel was considered an unusual and difficult task for that day, since it was dug through a bank of fine sand which was excavated by a jet of water and washed into a low portion of the lake front that lies at its west portal.

The family of D. C. Linsley was prominent and influential in public affairs. His father was a judge in the Vermont Supreme Court and his son, J. H. Linsley, M.D., established the Vermont Laboratory of Hygiene. Physically, Mr. Linsley was a tall, broad-shouldered man with a fine physique. He was dignified and austere but courteous and kindly in his dealings with others—"a gentleman of the old school."

Jasper O. Draffin, M. Am. Soc. C.E.
Associate Professor, Theoretical and Applied Mechanics, University of Illinois

Urbana, Ill.
August 10, 1932

Behavior of Earth Banks During Construction

Dear Sir: I have studied with much interest the informative article by Mr. Washbourne on "New Types of Box Piles," which appeared in the July issue. The part of this article that describes a method of introducing sufficient lateral stiffness into a pile so that it will help retain the exposed side bank again brings up the question, "What is the exact behavior of a side bank on which an old building rests and alongside which a new subway must be constructed?" In Proceedings for October 1931, Glennon Gilboy, Jun. Am. Soc. C.E., describes a machine designed to answer this question, and J. C. Meem, M. Am. Soc. C.E., in Proceedings for December 1931, describes another set of apparatus also designed to solve this problem. It has occurred to me that a full-sized field experiment dealing with undisturbed soil as found in field conditions would answer many questions about the forces affecting an underpinning and retaining wall, as described by Mr. Washbourne.

Such an experiment could be performed as follows (Fig. 1):

1. Drive two parallel rows of master piles.
2. Weld steel compression struts to them at their tops and place electric strain gages at the center of the compression struts to register compression.
3. Sink a sheeted pit at each end of the rows of piles to a depth of about 35 ft.
4. Drift in between the rows of piles (Drift 1) and place a row of steel compression struts, welded to the piles at their centers. Place strain gages on them also.
5. Continue the pits down to the subgrade of the piles, drift in between the piles (Drift 2), and place a third set of compression struts with strain gages on them at the bottoms of the piles.
6. Place an extra pile in a concrete base in each end pit and anchor this securely at the top to take care of the end reaction.
7. Excavate the material in between the rows of piles while sheeting from pile to pile with horizontal sheeting. Note the compression in the steel struts (1) during excavation, (2) over a considerable period of time, and (3) during different conditions of weather and surcharge.
8. Replace the steel struts with timber braces and allow movement to occur and note the behavior after movement has started.
9. Repeat this experiment in (1) sand and gravel, (2) sand and loam and (3) clay.

Of course this is only a brief suggestion but it decribes an experiment which would definitely show the behavior of a bank of undisturbed soil as found in field conditions.

It cannot be too thoroughly emphasized that every large underpinning operation presents this problem of the underpinning and retaining wall which generally holds back virgin soil and not

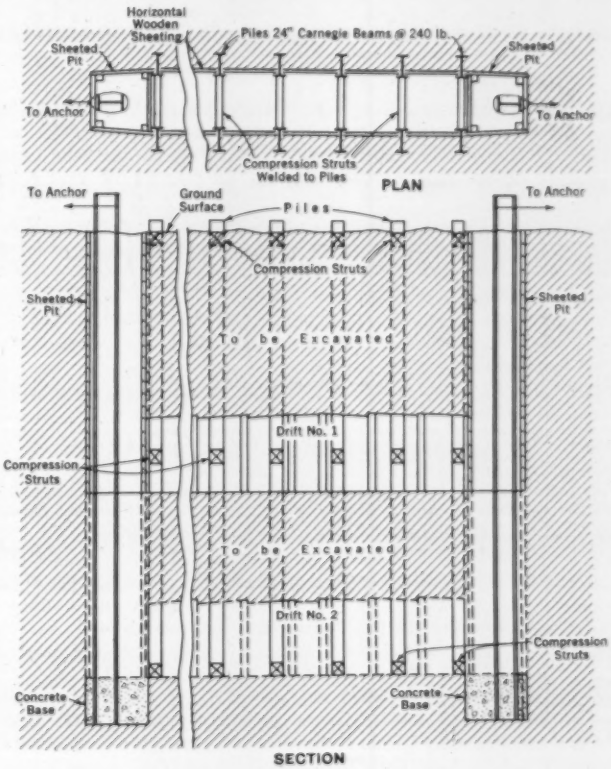


Fig. 1. Test to Determine Horizontal Earth Forces Under Field Conditions

backfill. In almost every case, this problem has to be met in a new manner, and it is of considerable value to engineers engaged in subway construction when these novel features are written up in articles such as Mr. Washbourne's.

Howard F. Peckworth, Assoc. M. Am. Soc. C.E.
Civil Engineer, Slattery-Daino Company, Inc.

Brooklyn, N.Y.
July 19, 1932

Valuation Terms Incorrectly Used

TO THE EDITOR: The paper by Mr. Black on "Present Status of Valuation Procedure," in the March issue, indicates that the influence of the decisions of the courts has all too frequently obscured fundamental principles of valuation. Attention will be called briefly to two matters.

The courts would apparently make the terms "value" and "rate-base" synonymous, in holding that value must be made the starting point when rates are to be determined. But value cannot be made the starting point because it is created by income, and income depends upon the rates. To some extent the resulting difficulty is overcome by interpreting "value" to mean not "value in exchange," but something else. The natural rate-base is, of course, the legitimately invested capital corrected for the relative purchasing power of the dollar and without diminution by accrued depreciation. This is of course not the only rate-base that can be used, although it is by far the most convenient. Furthermore, engineers and accountants should, wherever practicable, give preference to the term "replacement requirement" instead of "current depreciation."

The second matter relates to the frequently improper use of the term "obsolescence." Obsolescence cannot be foreseen. It cannot be taken into account before the event. Inadequacy can be foreseen. It involves replacement in combination with betterment, and provision should be made in the reserves for replacements to cover the resulting increment of operating cost. Obsolescence, on the other hand, refers to the discarding of parts of a plant while they are still in good condition, because the invention of a better machinery or of a new process of manufacture has made them superfluous.

An example of this is the relatively recent invention of the process which makes possible the production of illuminating gas from oil. This caused the scrapping of parts of plants that might otherwise have served for years. The sacrifice of capital, in such a case, must be made good afterward out of the increased earnings that result from reduced cost of operation. When a private company thus makes a sacrifice of capital, while at the same time reducing the cost of production, it should be allowed to amortize this loss fully. It should, however, also be willing to let the rate payer share the advantages derived from the invention by a reasonable reduction in rates. The warning, frequently given, not to confound amortization with inadequacy has not been heeded by many valuation engineers—therefore I venture this repetition.

C. E. GRUNSKY, PAST-PRESIDENT AM. SOC. C.E.
C. E. Grunsky Company, Engineers
San Francisco, Calif.
July 7, 1932

Washington's Street Layout

DEAR SIR: The article by Mr. Schwada on naming streets and numbering houses, in the March issue, rather surprisingly made no reference to the system long used in Washington, D.C. The scheme is so excellent, it merits description. The city is divided on cardinal lines by streets having a common point of origin at the Capitol. These are North, South, and East Capitol streets. Actually, West Capitol Street does not exist, because its place is taken by the Mall. These four street lines divide the city into four quadrants—northwest, northeast, southeast, and southwest (designated N.W., N.E., etc.).

The north and south streets paralleling North and South Capitol streets on each side of the city are numbered consecutively as far as necessary, and the numbered streets in a northerly direction may be remembered by their initial letter "N." East and west streets paralleling on each side the other coordinate streets are designated by the letters of the alphabet. Because the alphabet is not extensible, a second, third, and fourth alphabet of named streets must be used. The names of the second alphabet have their initial letters alphabeted and are of two syllables; similarly the names in the third alphabet have three syllables; and the fourth alphabet has botanical names. For instance, Allston Street is the first street of the second alphabet; Allison of the third; and Aspen of the fourth.

Beyond the first lettered alphabet the east and west streets are frequently discontinuous because of the independent development of subdivisions. However, this leads to no confusion whatever for all houses are numbered 100 to the block. Aspen Street crosses 9th Street; and between 8th and 9th streets is the 800 block of Aspen. Far to the west in another outlying development there may be an Alder Street—say between 43d and 44th streets. There the numbers are in the 43d block and the only numbers on Alder Street in such case would be from 4300 to 4399.

Generally, the city survey is so coordinated that the alphabets are controlled by zone bands so that Aspen and Alder will be approximately on a line and the same distance north of East and West Capitol streets, which together constitute their axis street. Thus, if one knows the number of a house, and the name, letter, or number of the street, together with the quadrant, which is designated in the usual way by letters, N.W., N.E., etc., the location is known. For example, 3819 Juniper Street, N.W., is between 38th and 39th streets at the letter "J" in the fourth alphabet.

The rectangular street grid in Washington is overlaid by a diagonal and irregular grid of avenues named after the states. The numbers on the avenues follow the same scheme and number as the named or numbered streets with which they are most nearly parallel. Thus, New York Avenue, which makes an angle of less than 45 deg. with the lettered streets, carries numbers like them; and 1308 New York Avenue, N.W., lies between 13th and 14th streets. One must know, however, where the avenues are, because 4309 New Mexico Avenue, N.W., does not locate the avenue in any alphabet; consequently one cannot tell how far out on 43d Street New Mexico Avenue will be found.

To one familiar with the Washington scheme, it is remarkably definite and convenient; and, of course, it may be extended just as far as ingenuity can provide names for more alphabeted streets. The fourth alphabet reaches the most remote line of the District of Columbia, as now constituted.

E. W. JAMES, M. Am. Soc. C.E.
Chief, Division of Design
Bureau of Public Roads
Washington, D.C.
August 8, 1932

Illustrating Geodetic Control

SIR: Control of geodetic precision is increasingly recognized as a necessity for all metropolitan areas. Once the precise initial values for latitude and longitude are established by the Federal Government, as outlined by Mr. Syme in the March issue, a system of traverse, or triangulation and traverse, may be established by the municipality. This local control survey establishes plane coordinates for points within the metropolitan area, the ultimate goal, of course, being to establish coordinates—first for every street intersection, angle, or curve point, and then for significant points upon boundaries of private properties.

Although there are a large number of coordinated points ultimately to be established, the building up of this great mass of information can be accomplished by a program extending over a considerable period of time, after the required initial monuments of the controlling traverses are established.

As soon as the required initial points are available from an adequate local control survey based upon Federal latitude and longitude, a city ordinance such as that recently passed in Toledo, Ohio, is an effective means of building up the coordinate system. This ordinance provides:

"Section 1. That the origin of plane coordinates of and for the City of Toledo be, and the same is hereby established, 131,482.0 ft. south and 118,923.0 ft. west of a point upon the Ohio Bank Building known as Triangulation Station Bank, the position of which upon North American datum is latitude 41° 39′ 04″.606 and longitude 83° 32′ 07″.360.

"Section 2. That the positions of the stations of the triangulation and of the first-order traverse in conjunction therewith, on file in the office of the Commissioner of Engineering, be accepted and their plane coordinate values construed as establishing the plane coordinate datum in their respective localities.

"Section 3. That all surveys affecting the position or improvement of streets and of public property shall be connected with, and adjusted to, the plane coordinate system thus established.

"Section 4. That the boundaries of all future recorded sub-

divisions and the angle and curve points of streets of such subdivisions be coordinated upon the plane coordinate datum of the City of Toledo, by means of connecting traverse surveys of an accuracy of at least one part in 10,000, said surveys to be connected with stations of the triangulation or of the first-order traverses at two or more points.

"Section 5. That all future ordinances or resolutions of the Council of the City of Toledo, containing descriptions of streets or of public property, shall include in such descriptions the plane coordinate positions of all angle and curve points."

Taking 37.2 miles as the width of a north-south belt within which a system of plane coordinates may be expanded without incurring errors greater than 1:100,000 (the present accepted base-to-base closure for city triangulation), it is obvious that the coordinates of any local plane coordinate system may be extended both east and west from a central origin for from 18 to 20 miles without serious difficulty. The cities of Albany and Schenectady, N.Y., for example, are located a distance of approximately 15 miles from center to center. Each has its own system of plane coordinates, the two origins being connected by triangulation. The reference meridian and the effective origin of each system are at the center of the city, but the mathematical origin is in each case located a sufficient distance south and west to ensure all points within the area having plus (north and east) coordinates.

All the surveys within each area are referred to the city's coordinate system. This not only provides the best possible basis for all local surveys but, since the two origins are interconnected by triangulation of geodetic precision, ensures that maps made upon either basis will match. Any point at the junction of the two survey areas may have two sets of coordinate values, but its actual position, as plotted upon any map projection or as computed in terms of latitude and longitude, will be the same.

R. H. RANDALL, M. Am. Soc. C.E.
President, R. H. Randall and Company

Toledo, Ohio
August 8, 1932

In Defense of Full-Width Roads

DEAR SIR: The paper by Mr. Catton in the August issue, in an effort to justify "half-a-loaf" expensive pavement slab in the improvement of local highways, appears to have omitted some of the important items of cost which seem essential to a comparison of this method with low-cost, full-width, all-weather surfaces. Certain fundamentals will prevail regardless of the final answer:

1. Every type of improvement will require maintenance to preserve structural integrity and riding quality.

2. Vehicle operating costs will be the same with an expensive pavement as with an adequate bituminous surface treatment—about half that of untreated surfaces.

3. Full-width (two-lane) pavement is superior to half-width pavement from the standpoint of safety, convenience, and appearance.

4. Half-portion improvement necessitates especial care and expense in shoulder construction. Untreated earth shoulders are not adequate for all-weather service.

5. Any half-portion improvement is of necessity a potential full-width highway, hence full-width grading, bridges, and drainage must be provided initially.

6. Hence, the problem resolves itself into a question of relative economy and utility between half-width expensive pavement and full-width, low-cost pavement, rather than between the former and untreated earth or gravel as cited in the paper.

7. The cost of applying and maintaining bituminous treatment to a road (such as gravel) is normally 40 per cent less than the cost of surface maintenance and unsalvable replacement of surfacing material (gravel) on an untreated surface; hence, if an improved road is justified, there is no sane excuse for not providing some form of adequate surface treatment. The operating saving effected by treatment is $3.65 a year per vehicle per day.

Obviously, high-cost one-lane construction is under a severe initial cost handicap as compared with low-cost full-width improvement. Full-width substantial improvements that are entirely adequate for traffic in excess of 1,000 vehicles per day are not at all unusual at a cost of $5,000 per mile, and seldom cost more than $10,000 per mile. Furthermore, such pavements are susceptible of economical perpetuation and improvement as traffic may require.

Aside from its added safety and convenience, the low-cost, whole pavement may be had at an initial cost less than, and even one-half that of a single-lane improvement, neglecting entirely the essential, but by no means insignificant, expense of a passing track with its necessary surface treatment and maintenance.

To one using the half-paved road the year round, the item of adequate passing lane becomes very important even though he has to turn out only two or three times in a few miles. A mired truck or car that has skidded into the side ditch is more than a mere item of expense; it may cause much wear and tear on human nature.

Maintenance expense for low-cost, all-weather surfaces is not high. It compares very favorably with similar costs common to portland cement concrete slabs (in many cases is less), and when the cost of maintaining the passing lane is added to the cost of maintaining the half-pavement, little if any advantage can be found in favor of the high-cost type.

As between the unimproved and the half-portion pavement, one would certainly choose the latter. But, in comparison with the available variety of low-cost types, suited to such a wide range of local conditions, the figures given and examples cited in Mr. Catton's paper are not very convincing.

H. W. SKIDMORE, Assoc. M. Am. Soc. C.E.
President, Chicago Testing Laboratory, Inc.

Chicago, Ill.
August 10, 1932

Highway Lighting Important

TO THE EDITOR: The excellent article on "Safety Islands—Their Design and Use" by Mr. Mickle, in the July issue, includes a description of a "protector" designed to minimize the effect of vehicle collisions. I do not believe, however, that he has emphasized sufficiently the important part that lighting plays in reducing the number of collisions. The first thought in safety-zone design should be the elimination, so far as possible, of accidents. Reduction in the severity of accidents is a secondary consideration.

The Detroit traffic survey of 1927 investigated 65 collisions at 49 protected safety zones, which occurred in the first four months of 1927. These accidents resulted in 88 injuries and 2 fatalities to occupants of vehicles. Only two of these accidents occurred in daylight, two at dusk, and 61 during hours of darkness. The greatest number took place between 2 and 3 a.m. when traffic is usually at a minimum. From this study one cannot fail to be convinced that inefficient lighting was a major causative factor, even if the lowered mental and physical efficiency of motorists during the late night and early morning hours is taken into consideration.

To be most effective, safety-zone lighting should be of four distinct types:

1. A steady or flashing amber light should be installed at a height at least 9 ft. above the pavement, making the presence of the obstruction visible to approaching vehicles over the tops of other cars at a considerable distance. Lights of this character are illustrated in the photographs in Mr. Mickle's article, showing safety zones in Cleveland, Detroit, and Cincinnati.

2. Amber warning lights should be installed about on a level with the driver's eyes. These may be either steady or flashing.

3. The side of the safety-zone abutment facing approaching traffic ought to be adequately flood-lighted.

4. Flood lights illuminating the pedestrian area are also useful, but probably not so necessary as the other three types. A light of this character is shown in Mr. Mickle's illustration of a safety zone in Columbus.

Reflector warning lights are sometimes used in addition to, or in substitution for, the lower warning signal. When used to augment the lighting arrangement previously described, they are probably most valuable in marking the outer limits of the zone.

HAWLEY S. SIMPSON, Assoc. M. Am. Soc. C.E.
Research Engineer, American Electric Railway Association

New York, N.Y.
August, 1, 1932

Traffic-Separation Plan for Two Intersecting Roadways

DEAR SIR: After reading the article on "Traffic Circles and Rotary Traffic" by Mr. Swan, in the July issue, I should like to call attention to a traffic-separation plan suitable for two intersecting roadways, as shown in Fig. 1. This plan can be used to advantage in many cases in which the acquisition of property along the two intersecting highways is expensive, and in other cases where the nature of the terrain is such that no widespread area is available. It is superior to the traffic circle of small radius in that no through traffic flow lines are intersected. Such a layout lends itself particularly to the condition in which a majority of the vehicles make a direct crossing, and provides ample space for the congestion of vehicles making right or left turns.

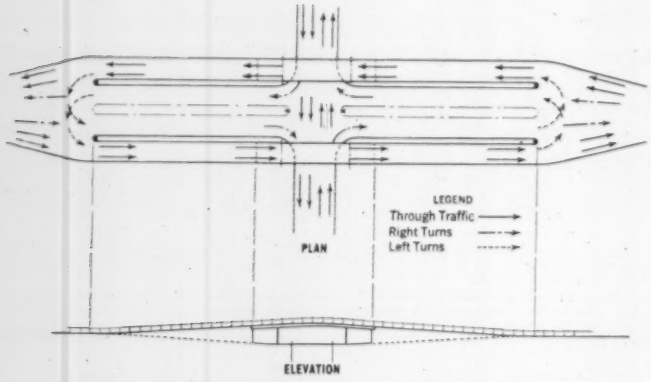


FIG. 1. TRAFFIC SEPARATION PLAN DESIGNED FOR TWO INTERSECTING ROADWAYS
No Through Traffic Flow Lines Intersected

If a conventional three-span continuous bridge made up of rolled steel sections is used, the minimum difference in elevation between the two levels is produced, and the angle of vision can be made quite ample. For instance, in a 68-ft. main span with 25.5-ft. side spans, 30-in. rolled beams topped with a concrete slab will carry an H-20 loading.

Although I do not know of any case where a similar layout has been used, I believe that this plan has merit.

CLARENCE D. FOIGHT, Assoc. M. Am. Soc. C.E.

Pittsburgh, Pa.
July 13, 1932

Suggested Tests of Statical Determinateness

DEAR SIR: The paper, "Bridges with Continuous Girders," by Mr. Lindenthal, in the July issue, suggests a subject deserving of further consideration—that is, the stability of a truss, the configuration of which contains polygons of four or more sides, using the term "truss" to include all planar structures which are pin-connected or are commonly considered to be so in computing primary stresses.

This question of stability is suggested by the truss shown by Type E in Fig. 4 of Mr. Lindenthal's article. I have sketched (see Fig. 1) four trusses of this type—greatly simplified for purposes of illustration. These trusses differ only in the length of certain panels which is $\sqrt{2}\,p$ instead of p, yet two of the trusses, B and D, are statically determinate and stable, while the other two are statically indeterminate and unstable. The instability of Trusses A and C may be readily shown by pin-connected cardboard models or by attempting to compute the bar stresses.

So far as I know, a general statement of the conditions that must be fulfilled by a statically determinate planar structure has not been published in American engineering literature. As commonly given, the statement is: if k is the number of joints and $2k$ is the number of unknowns (bar stresses and reaction components), the structure is statically determinate. Although the converse is true, the foregoing statement is not sufficient. Each of the structures shown in Fig. 1 has 18 joints and 36 unknowns, yet two of the trusses are not statically determinate. As shown by Müller-Breslau (in *Die Graphische Statik der Baukonstruktionen*, 1905 ed., Vol. 1, page 439), it is also necessary that the $2k$ simultaneous equations be satisfied by one and only one set of values of the unknowns—that is, the equations must be independent and consistent. He tests the fulfillment of this condition by letting the $2k$ equations ($\Sigma V = 0$ and $\Sigma H = 0$ at each joint) be represented by

$$\begin{aligned} a_1x_1 + a_2x_2 + \ldots + a_{2k}x_{2k} &= P_1 \\ b_1x_1 + b_2x_2 + \ldots + b_{2k}x_{2k} &= P_2 \\ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots & \\ m_1x_1 + m_2x_2 + \ldots + m_{2k}x_{2k} &= P_m \end{aligned}$$

where the x's are the unknowns, the P's are the components of the external loads applied at the joints, and the coefficients of the x's are functions of the slopes of the lines of action of the unknowns. Let D be the determinant of the coefficients of the unknowns.

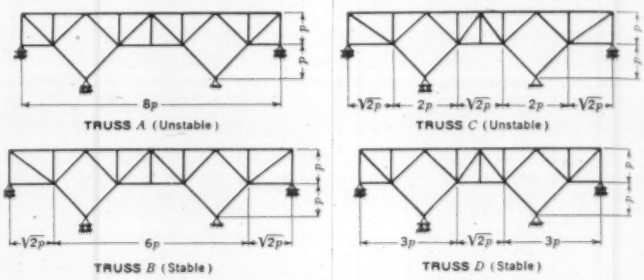


FIG. 1. SIMILAR TRUSSES THAT HAVE DISSIMILAR STATICAL PROPERTIES

$$\begin{vmatrix} a_1a_2 \ldots \ldots a_{2k} \\ b_1b_2 \ldots \ldots b_{2k} \\ \ldots\ldots\ldots\ldots \\ m_1m_2 \ldots \ldots m_{2k} \end{vmatrix}$$

Then, from the theory of simultaneous equations, if D is different from zero, the equations are independent and consistent. The complete theorem of Müller-Breslau may be stated as follows: a planar structure of k-joints is statically determinate (and stable) if there are $2k$-unknowns and $D \lesseqgtr O$. If $D = O$, there are two possibilities—either there is an indefinite number of solutions or there is no solution. However, in most if not all cases, a structure that satisfies the first condition but has $D = O$, will be found to have no solution. The fact that no solution exists for the equations of equilibrium means that equilibrium cannot exist and the structure is therefore unstable. This can be readily shown by attempting to find the bar stresses, by a model or by a study of the displacements when the structure is treated as a kinematic chain.

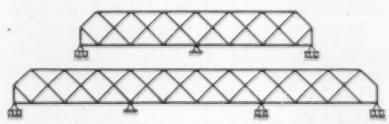


FIG. 2. TYPES OF "RAUTEN" TRUSSES

To the structures shown in Fig. 4 of Mr. Lindenthal's paper, I should like to add the "Rauten" trusses, shown in Fig. 2 herewith. The two-span structure is statically determinate; the three-span structure is statically indeterminate to the first degree.

ALFRED T. WAIDELICH, Jun. Am. Soc. C.E.

Assistant Instructor in Civil Engineering
Massachusetts Institute of Technology

Philadelphia, Pa.
August 10, 1932

Fall Meeting, Atlantic City, N.J.

Four Days, October 5-8, 1932, in and Around Famous Shore Resort Offer Attractive Sessions and Social Events

General Meeting—Philadelphia Tri-State Improvements

WEDNESDAY—October 5, 1932—Morning

9:00 **Registration**

Lounge Floor

GENERAL MEETING

VERNON ROOM, LOUNGE FLOOR

10:00 **Fall Meeting called to order by**

CLARENCE E. MYERS, *M. Am. Soc. C.E., Consulting Engineer; President, Philadelphia Section Am. Soc. C.E.*, Philadelphia.

10:05 **Addresses of Welcome**

His Excellency, A. HARRY MOORE, *Governor of the State of New Jersey*

His Honor, HARRY BACHARACH, *Mayor of Atlantic City, N.J.*

10:30 **Response**

HERBERT S. CROCKER, *President, American Society of Civil Engineers; Consulting Engineer*, Denver, Colo.

10:45 **Improvement of Railroad Passenger Facilities in Philadelphia**

ROBERT FARNHAM, *M. Am. Soc. C.E., Chief Engineer, Philadelphia Improvements, Pennsylvania Railroad Company*, Philadelphia.

12:30 **Buffet Luncheon for Members**

Rutland Room (First sleeping-room floor). Tickets $1.00 each.

12:30 **Buffet Luncheon for Ladies**

Benjamin West Room (Meeting Room floor). Tickets $1.00 each.

Advance Registration

For the convenience of those who plan early to attend the meeting and to facilitate the making of reservations for rooms and meeting events, an advance registration blank is provided on page 13 of the advertising section. By filling out and sending in this form you will greatly expedite the task of the Local Committee in preparing for your entertainment.

FOR ADVANCE REGISTRATION FORM

Turn to Page 13 →

WEDNESDAY—October 5, 1932—Afternoon

GENERAL MEETING

VERNON ROOM, LOUNGE FLOOR

THE REGIONAL PLAN OF THE PHILADELPHIA TRI-STATE DISTRICT, A SYMPOSIUM

2:30 **Development of the Regional Planning Movement in the Philadelphia Tri-State District**

SAMUEL PRICE WETHERILL, JR., *Esq., Former President, Regional Planning Federation of the Philadelphia Tri-State District*, Philadelphia.

3:00 **Public Relations**

HERBERT L. BADGER, *Esq., Vice-President, Bell Telephone Company of Pennsylvania, Vice-President, Planning Federation of the Philadelphia Tri-State District*, Philadelphia.

3:30 **Making the Plan and Writing the Report**

WILLIAM H. CONNELL, *M. Am. Soc. C.E., Executive Director, Regional Planning Federation, Philadelphia Tri-State District*, Philadelphia.

4:00 **Opening Discussions**

The Regional Plan in New Jersey

HENRY J. SHERMAN, *M. Am. Soc. C.E., Consulting Engineer, New Jersey Board of Commerce and Navigation; Consulting Engineer*, Camden, N.J.

The Regional Plan in Delaware

ALBAN P. SHAW, *Esq., County Engineer, New Castle County*, Wilmington, Del.

WEDNESDAY—October 5, 1932—Evening

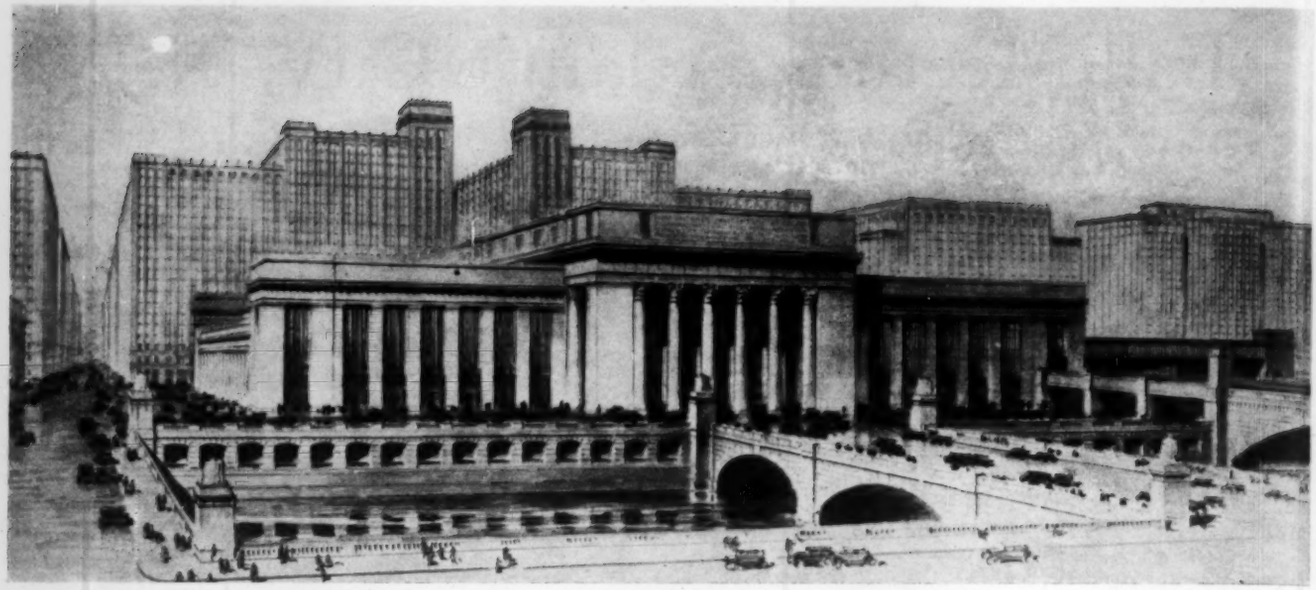
7:00 **Dinner**

Rutland Room (First sleeping-room floor).

Following dinner there will be addresses on problems of the day. Among the speakers will be ELISHA LEE, *Vice-President, Pennsylvania Railroad Company*, Philadelphia; and CONRAD N. LAUER, *President, American Society of Mechanical Engineers*, Philadelphia.

Tickets for the dinner are $5.00 each. Informal evening dress will be in order.

FALL MEETING HEADQUARTERS—CHALFONTE-HADDON HALL HOTEL



PENNSYLVANIA RAILROAD STATION, 30th STREET, PHILADELPHIA

Time Allotted Both to Technical Divisions and Golf

THURSDAY, October 6, 1932—Morning

SESSIONS OF TECHNICAL DIVISIONS

9:00 **Registration**

Mandarin Room (Meeting Room floor)

SANITARY ENGINEERING DIVISION

BENJAMIN WEST ROOM

9:30 **The Washington, D.C., Refuse Problem**

H. P. EDDY, JR., *M. Am. Soc. C.E., Consulting Engineer*, Boston, Mass.

10:00 **Discussion opened by**

J. B. GORDON, *M. Am. Soc. C.E., Sanitary Engineer, District of Columbia*, Washington, D.C.

10:15 **The Joint Trunk Sewer for the Hackensack District**

GEORGE W. FULLER, *M. Am. Soc. C.E., Consulting Engineer*, New York, N.Y.

10:45 **Discussion opened by**

THOMAS J. WASSER, *Assoc. M. Am. Soc. C.E., Chairman, Hackensack Valley Sewerage Commission (Bergen and Hudson Counties)*, Jersey City, N.J.

HARRY P. CROFT, *Esq., Chief Engineer, State Department of Health*, Trenton, N.J.

11:15 **Sewage Disposal of the New Jersey Coast Resorts**

CHESTER G. WIGLEY, *M. Am. Soc. C.E., Consulting Sanitary Engineer*, Atlantic City, N.J.

11:45 **Discussion opened by**

ALMON L. FALES, *M. Am. Soc. C.E., Consulting Engineer*, Boston, Mass.

CLYDE POTTS, *M. Am. Soc., C.E., Consulting Engineer*, New York, N.Y.

JOHN R. VAN DUYNE, *M. Am. Soc. C.E., Chief Engineer, Passaic Valley Sewerage Commissioners*, Newark, N.J.

CITY PLANNI. G DIVISION—SURVEYING AND MAPPING DIVISION

VIKING ROOM

9:30 **Types of Surveys Used in Regional and City Planning and the Reasons for Using Them**

RUSSELL V. BLACK, *M. Am. Soc. C.E., City Planner*, New Hope, Pa.

10:00 **The City Plan—a Summarization of What It Should Include**

E. P. ARNESON, *M. Am. Soc. C.E., Civil Engineer*, San Antonio, Tex.

10:45 **Discussion opened by**

ROSCOE N. CLARK, *M. Am. Soc. C.E., City Engineer and Manager, Bureau of Public Works, Metropolitan District Commission*, Hartford, Conn.

E. P. GOODRICH, *M. Am. Soc. C.E., Consulting Engineer*, New York, N.Y.

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COORDINATION OF AIR AND HIGHWAY TRAFFIC

The Central Airport in Camden, N.J., Is One of the Busiest in the United States

SEAVIEW GOLF CLUB AT ABSECON, N.J.
All the Facilities of this Famous Club Are Available to Members and Guests

THURSDAY—October 6, 1932—Afternoon

GOLF AND OTHER DIVERSIONS

2:00 **Golf Tournament** at the Seaview Golf Club, Absecon, N.J., for members and guests.

For those who do not "golf" there are boating, fishing, cruising, and the many other attractions of Atlantic City to occupy the afternoon.

Doubtless parties may be formed to renew old, and make new friendships.

Members wishing to arrange for boating and fishing, etc., for which arrangements must be made in advance, should notify the Registration Desk as early as possible of their plans.

Buses leave the Hotel at 1:15 p.m. for the golf tournament. The greens fees are $3.00 per person, payable at the golf club.

Members and their ladies attending the Meeting may enjoy the facilities of the Seaview Golf Club or other golf clubs at any time and are expected to settle their own accounts as they are incurred. Cards of admission to the clubs may be had on application at the Registration Desk.

ENTERTAINMENT FOR THE LADIES

1:00 **Luncheon and Bridge** for the ladies at the Seaview Golf Club, Absecon, N.J.

The ladies will be entertained at luncheon and bridge as guests of the Philadelphia Section with the Ladies Reception Committee as hostesses.

Those ladies who prefer golf to bridge may join in the men's tournament, which starts at 2:00 p.m.

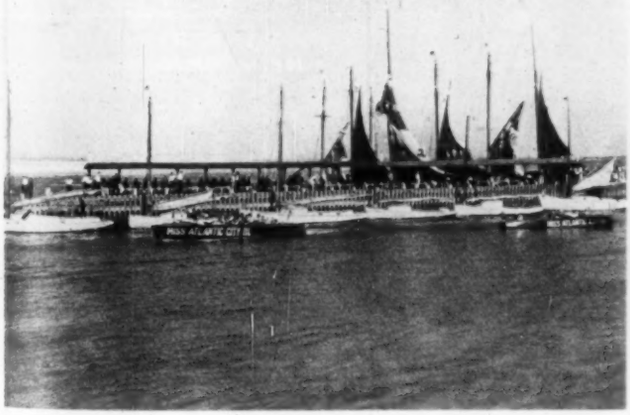
The Seaview Golf Club is renowned for its course and its clubhouse facilities, all of which have been placed at the disposal of Society visitors.

THURSDAY—October 6, 1932—Evening

9:30 **Reception, Entertainment, and Dance**

This function will be held in the Vernon Room (Lounge Floor) and there will be no charge to members and guests.

The early evening has been left open so as to permit the renewal of acquaintances, small dinner parties, etc.



FISHING BOATS IN THE INLET
Boating and Fishing Parties May Be Arranged

AN ATLANTIC CITY BEACH SCENE
Surf Bathing, a Major Attraction at the Resort

Entertainment and More Technical Sessions

***FRIDAY—October 7, 1932—Morning*—SESSIONS OF TECHNICAL DIVISIONS**

9:00 Registration

Mandarin Room (Meeting Room floor)

SANITARY ENGINEERING DIVISION

BENJAMIN WEST ROOM

9:30 Problems of Water Supply, Sewerage and Industrial Waste Disposal in Delaware County, Pennsylvania

H. M. FREEBURN, *Assoc. M. Am. Soc., C.E., District Engineer, Pennsylvania Department of Health*, Philadelphia.

10:00 Dicussion opened by

GEORGE D. NORCOM, *Assoc. M. Am. Soc. C.E., Sanitary Engineer, Federal Water Service Corporation*, New York.

10:30 Ground Water Resources of New Jersey

HOWARD T. CRITCHLOW, *M. Am. Soc. C.E., Division Engineer, State Water Policy Commission*, Trenton, N.J.

11:00 Discussion opened by

JOHN N. BROOKS, *M. Am. Soc. C.E., Assistant Division Engineer, State Water Policy Commission*, Trenton, N.J.

JOHN C. HOYT, *M. Am. Soc. C.E., Hydraulic Engineer in Charge of Surface Water Division, U.S. Geological Survey*, Washington, D.C.

DAVID G. THOMPSON, *Esq., Senior Geologist, U.S. Geological Survey*, Washington, D.C.

11:30 General Discussion of Sanitary Engineering Division Activities

HIGHWAY DIVISION

VIKING ROOM

9:30 Traffic Control from the Standpoint of the State Highway System

J. L. BAUER, *M. Am. Soc. C.E., State Highway Engineer, State Highway Commission*, Trenton, N.J.

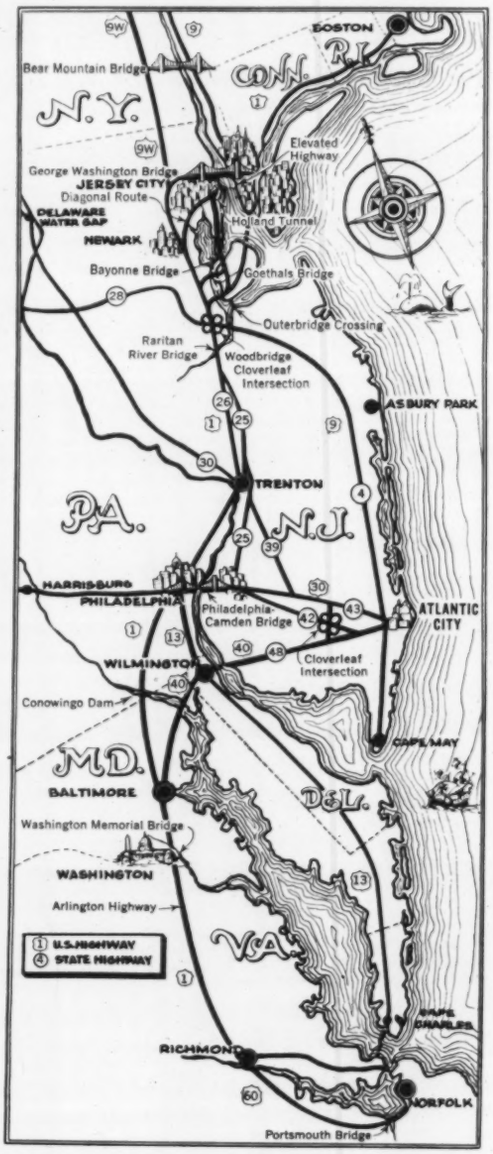
10:00 Traffic Control from the Standpoint of City Traffic

BURTON W. MARSH, *M. Am. Soc. C.E., Traffic Engineer, Department of Public Safety*, City of Philadelphia.

10:30 Can the Highway Investment Be Justified on Economic Grounds?

T. R. AGG, *M. Am. Soc. C.E., Dean, College of Engineering, Iowa State College*, Ames, Iowa.

11:00 Discussion



AUTOMOBILE ROUTES TO AND AROUND ATLANTIC CITY

A Network of Fine Highways Makes Sightseeing by Motor Attractive and Easy

ENGINEERING-ECONOMICS AND FINANCE DIVISION

TOWER ROOM

9:30 The Relation of Investment in Capital Facilities and Construction to the Economic Scheme

JOHN P. HOGAN, *M. Am. Soc. C.E., Consulting Engineer* New York, N.Y.

10:00 Public Works Financing

MALCOLM PIRNIE, *M. Am. Soc. C.E., Consulting Engineer*, New York, N.Y.

10:30 Combining Compulsory Corporate Reserves and Public Works Construction for National Economic Stability

FREDERICK H. MCDONALD, *M. Am. Soc. C.E., President, McDonald and Company, Engineers and Architects*, Atlanta, Ga.

11:00 Discussion

FRIDAY—October 7, 1932—Afternoon

2:00 Student Chapter Conference

VIKING ROOM

Representatives of the Student Chapters in the Region will meet for round table discussion of mutual problems.

Boating, Fishing, Sightseeing, Golf, Motoring

This afternoon is otherwise free for members, ladies, and guests to take advantage of the opportunity to visit and enjoy the many attractions of Atlantic City. Those who wish to go on boating and fishing trips and the like, should notify the Registration Desk in advance so that the necessary arrangements can be made.

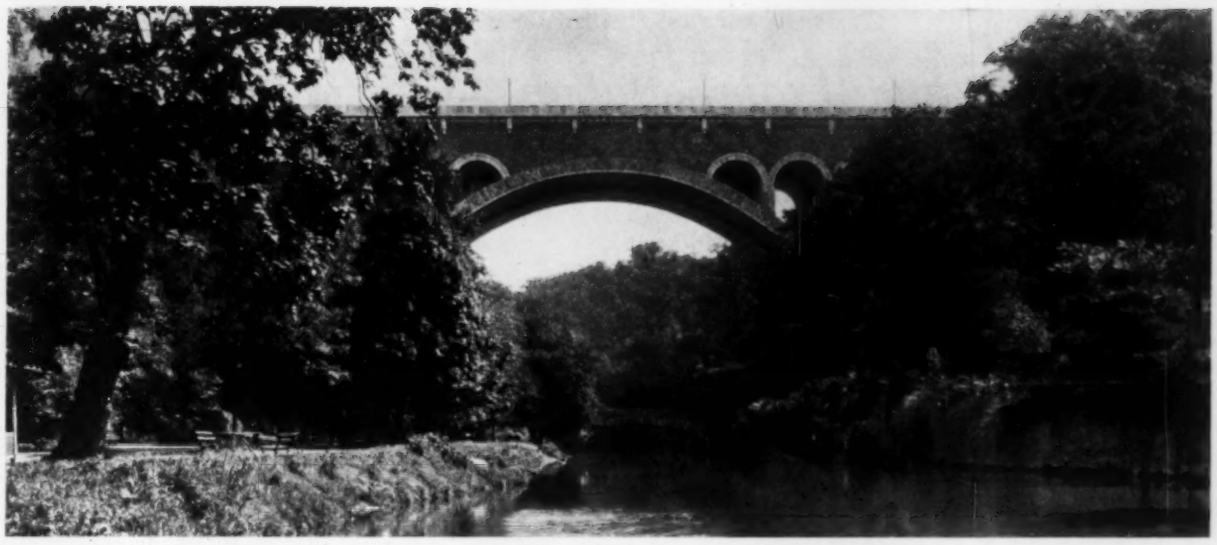
If a sufficient number so desire, arrangements will be made for a visit to the U.S. Naval Air Station at Lakehurst, N.J., where the hangar for the *Los Angeles* and *Akron* is located.

Afternoon Tea for the Ladies

IN THE LOUNGE

Tea for the ladies will be served in the Lounge from 4:30 p.m. to 5:30 p.m.

On the Meeting Room floor, Room 1344 is especially reserved for the use of the ladies as a meeting place both during the day and in the evening.



HENRY AVENUE BRIDGE OVER WISSAHICKON CREEK, FAIRMONT PARK, PHILADELPHIA

Philadelphia Claims Full Day for Sightseeing

SATURDAY—October 8, 1932—Morning and Afternoon

8:45 Departure for Philadelphia

The party will depart from Atlantic City by train, special Pullmans being provided through the courtesy of the Philadelphia Section. Members will pay only rail fare.

10:00 Arrival at 30th Street Station of the Pennsylvania Railroad, Philadelphia

Members will be escorted on an inspection trip to the Philadelphia improvements under construction nearby, which have many attractions for engineers. The ladies, and members who wish, will be taken by bus on a trip to points of interest in Philadelphia, which is noted for its historic sites and its beautiful parks. Among the historic shrines may be mentioned the grave of Benjamin Franklin; Independence Hall, containing the Liberty Bell and the room in which the Declaration of Independence was signed; the Betsy Ross House, where the first flag adopted by the United States was made; Continental Hall, in which the First Continental Congress assembled in 1774; and Congress Hall, in which President Washington was inaugurated for his second term.

12:30 Luncheon

Members, ladies, and guests will assemble in the new 30th Street Station of the Pennsylvania Railroad.

2:00 Bus Trip

Following luncheon, the party will leave the 30th Street Station by bus for a trip to Valley Forge, passing en route some of Philadelphia's fine residential sections. Valley Forge, another of the country's historic shrines, contains Washington's Headquarters, the remains of the original trenches and other features, including the Memorial Chapel.

There will be no charge to members and guests for the special Pullman service, the luncheon, and the excursions in the morning and afternoon, these features being provided through the courtesy of the Philadelphia Section.

On return to the 30th Street Station about 5:30 p.m., the Fall Meeting will be officially at an end.

NOTE: *Members returning to Atlantic City after the Valley Forge trip can take trains from the 30th Street Station. Members who will not return to Atlantic City should bring their baggage and check it at the 30th Street Station.*

MEMORIAL ARCH AND WASHINGTON'S HEADQUARTERS AT VALLEY FORGE

Hotels and Transportation—Official Notices

REDUCED RAILROAD RATES

Application has been made for reduced rates on the certificate plan. Half rates for the return trip are contingent on the validation of a minimum of 100 certificates. All fares in excess of 67 cents come under this plan.

Members or others who attend the Fall Meeting should purchase one-way tickets, and obtain from the ticket agent a certificate for each ticket purchased, for validation at the Registration Desk at Meeting Headquarters.

Do not fail to obtain a certificate for each ticket purchased.

IMPORTANT

Make early inquiry of your ticket agent in regard to the selling dates of tickets. Purchase your tickets within the selling dates for your territory.

HOTEL RATES

American Plan (Room and Meals)

	With Private Bath		Without Private Bath	
	One Person	Two Persons	One Person	Two Persons
Chalfonte-Haddon Hall	$7.00–10.00	$13.00–18.00	$6.00–8.00	$12.00–14.00
Hotel Morton	6.00– 7.00	10.00–12.00	5.00 up	9.00 up
Marlborough-Blenheim	6.00–10.00	10.00–20.00	.....	.....
Cheltenham-Revere	5.00 up	10.00 up	4.00 up	8.00 up
Hotel Traymore	8.00 up	12.00 up	.....	.....
Hotel Dennis	6.50– 9.00	12.00–20.00	6.00 up	11.00–13.00
Glaslyn-Chatham	5.00– ...	10.00– ...	4.00	7.00– 9.00

European Plan (Room Only)

	With Private Bath		Without Private Bath	
	Single Room	Double Room	Single Room	Double Room
Chalfonte-Haddon Hall	$3.00– 6.00	$5.00–10.00	.....	.....
Hotel Morton	3.00 up	5.00– 6.00	2.00 up	4.00 up
Marlborough-Blenheim	3.00– 7.00	4.00–14.00	.....	.....
Cheltenham-Revere	3.00 up	6.00 up	2.00 up	4.00 up
Hotel Traymore	4.00–10.00	6.00–12.00	.....	.....
Hotel Dennis	3.50– 6.00	5.00–12.00	3.00 up	4.00– 6.00
Glaslyn-Chatham	3.50 up	5.00– 6.00	2.50 up	4.00– 5.00

The Chalfonte-Haddon Hall is the Meeting Headquarters and, it is expected, will be able to furnish accommodations for all visitors. Members and guests in attendance at the Meeting who stop at the Chalfonte-Haddon Hall on the American Plan will receive credit, when paying their bill, of $1.00 for the buffet luncheon on Wednesday and $2.00 for the dinner on the same day, on presentation of ticket stubs.

Reservations are to be made directly with the Chalfonte-Haddon Hall, Atlantic City, N.J., and should be engaged as early as possible in order to avoid delay and inconvenience in obtaining accommodations on arrival.

ENTERTAINMENT FOR THE LADIES

Attention is directed to the program of entertainment for the ladies. The Ladies' Reception Committee has planned to entertain visiting ladies in a delightful manner that will make the Meeting long remembered.

Tea for the ladies will be served every afternoon at 4:30 p.m. to 5:30 p.m. in the lounge. Room 1344, on the Meeting Room floor, is reserved each day and evening as a meeting place for ladies.

INVITATION TO STUDENT CHAPTERS

All members of Student Chapters are cordially invited to attend and participate in all events of the Fall Meeting.

GOLF

On application at the registration desk, golf privileges at the Seaview and other golf clubs will be extended to members and other guests attending the Meeting, subject to payment of the usual greens fees and other charges incurred.

ORDER ALL TICKETS IN ADVANCE

Members who order tickets in advance will find tickets and badges awaiting them on arrival, and will greatly assist the Local Committee by giving advance information to guide it in concluding the arrangements called for in the program.

Registration coupons and orders for tickets should be sent to C. S. Shaughnessy, Room 975, City Hall, Philadelphia, Pa.

See page 13 for advance registration coupon. ⟶

LOCAL COMMITTEES ON ARRANGEMENTS

EXECUTIVE COMMITTEE

F. O. Dufour, *General Chairman*

J. W. Follin, *Assistant General Chairman*

C. A. Howland, *Secretary*

C. E. Myers, *President, Philadelphia Section*

Robert Farnham, T. P. Watson, Clark Dillenbeck, Henry J. Sherman, William Easby, Jr., Benjamin Franklin, John Meigs, W. R. Scanlin, S. M. Swaab, C. S. Shaughnessy, H. W. Latta, E. H. Rockwell, Charles H. Stevens, C. L. Warwick, and F. H. Snow

Ladies' Reception Committee: Mrs. Henry J. Sherman, chairman; Mrs. Robert Farnham vice-chairman; Mrs. A. H. Nelson, Mrs. R. W. Randolph, Mrs. C. G. Wigley, Mrs. J. H. M. Andrews, Mrs. G. Schobinger, Mrs. F. O. Dufour, Mrs. Clark Dillenbeck, Mrs. J. W. Follin, Mrs. W. R. Scanlin, and Mrs. Charles H. Stevens

Finance Committee: Charles H. Stevens, chairman; Benjamin Franklin, C. E. Myers, W. R. Scanlin, and Henry J. Sherman

Activities Committee: T. P. Watson, chairman; C. L. Warwick, W. K. Brownell, C. L. Appleton, Lyle Jenne, Don Sommerville, and E. B. Temple

Transportation Committee: William R. Scanlin, chairman; T. P. Watson, A. H. Nelson, J. L. Bauer, R. G. Cone, W. Sebastian, and L. Lessig

Information and Service Committee: C. A. Howland, chairman; C. G. Wigley, A. Thomassen, and W. H. Gibson

Publicity Committee: Clark Dillenbeck, chairman; Robert Farnham, F. O. Dufour, J. W. Follin, L. Focht, C. A. Howland, Charles H. Stevens, Henry J. Sherman, F. E. Schall, C. L. Warwick, F. H. Snow, and H. W. Latta

Hotels Committee: H. W. Latta, chairman; Richard W. Randolph, Marion den H. Kolyn, Frank M. Masters, and Beale M. Schmucker

Registration Committee: C. S. Shaughnessy, chairman; H. M. Freeburn, P. A. Beatty, P. H. Carlin, S. Harris, H. L. Kinsel, W. I. Lex, E. H. Maier, T. L. McNee, L. F. Parlette, L. A. Raymond, W. I. Risley, W. E. Rosengarten, F. G. Schworm, J. W. Silliman, W. N. Collinson, R. G. Develin, J. Duncan Jacques, and Harry Robinson

Technical Meetings and Discussions Committee: Henry J. Sherman, chairman; S. M. Swaab, William H. Connell, J. L. Bauer, C. E. Chase, S. W. Sawin, Charles H. Stevens, J. W. Follin, John Meigs, and H. T. Critchlow

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The program has been prepared by the Local Committee under the direction of the Northern Regional Meeting Committee: H. M. Waite, Vice-President Am. Soc. C.E., chairman; Henry R. Buck, M. L. Enger, Robert Hoffmann, Edward P. Lupfer, E. K. Morse, Henry E. Riggs, and Charles H. Stevens, Directors Am. Soc. C.E.

Please call on the members of the Local Committee or on the Secretary's Office for any service desired.

Use Railroad Certificate to Obtain Reduced Fare. Consult Your Ticket Agent for Selling Date in Your Territory.

SOCIETY AFFAIRS

Official and Semi-Official

Researches in Meteorology Needed

SOCIETY COMMITTEE OFFERS INTERESTING SUBJECTS FOR THESES AND OTHER STUDENT USE

FOR THE PAST year or more, the Society's Committee on Meteorological Data has been studying ways and means whereby the work of the U.S. Weather Bureau may be made of greater service to the engineering profession. In the replies to a questionnaire sent to about two hundred engineers and others, much emphasis was laid on the lack of research work in meteorology of use to the profession, and on specific activities to yield results of value.

Thus the committee has conceived the idea of instituting a program of research for senior and graduate engineering students. It would fall into two types:

1. Analysis of existing data now published by the Weather Bureau, which might be done by one or more senior or graduate students in one year; and
2. Collection of new data over an extended period, say five to ten years, and its analysis, in a series of theses, the final results to be given in a graduate thesis.

The Society's committee would serve as a coordinating agency in working out a comprehensive program. Members of the Society particularly interested in certain phases would be assigned in an advisory capacity in connection with all work on a given subject. A copy of each thesis would be filed with the committee, and when the work on a specific subject was considered completed, all theses on this subject would be analyzed and a final report made by the committee.

Doubtless some of the suggested lines of research would involve the expenditure of considerable sums for equipment and apparatus, but other work could be carried on with little outlay. Schools might arrange with manufacturers of recording apparatus, such as rain gages, thermometers, and possibly anemometers and barographs, for loans or donations of such instruments. Also, some apparatus might be made by students.

SUGGESTED INVESTIGATIONS

The following lines of investigation are suggested as involving little outlay for apparatus:

1. Precipitation
   - *a.* Relation between results from gages on top of buildings and those at ground level. The location of many Weather Bureau gages has been changed frequently during the period of their record. It is a well established fact that there is considerable difference in catch at the two exposures, the amount decreasing with the height of the building. New records of gages stationed at previous Weather Bureau locations would enable former records to be reduced to a common denominator.
   - *b.* Effect on catch of exact location of gage on top of high buildings. Scattered experiments indicate considerable variation for different points on the same building.
   - *c.* Effect of buildings, trees, fences, etc., on the observed catch in gages. Present practice is to locate the gage so that lines drawn 45 deg. above the horizon from the top of the gage will clear any object. Results from a series of gages at various distances from buildings, fences, trees, etc., would prove whether this slope is sufficiently flat.
   - *d.* Relation between the amount of precipitation caught by rain gages of various diameters and that registered by the standard U.S. Weather Bureau gage; also that by such homely equipment as tomato cans, washtubs, buckets, etc. Many such records from isolated locations are open to question.
   - *e.* Comparison of catch of gages read infrequently, such as once a month or once every six months, with that of those read daily. In isolated localities it is difficult to secure frequent readings, and to judge the amount of evaporation. Some experimenters have used a small amount of olive oil to prevent evaporation.
   - *f.* Effect of elevation. There is a general impression that precipitation increases when the prevailing winds are at right angles to the face of a mountain range. The effects of slope, topography, etc., have never been investigated to any great degree.
   - *g.* Effect of irregular topography on precipitation and resulting isohyetals.
   - *h.* Interception of precipitation by vegetation. Not only trees, but also brush and grasses should be studied for both heavy and light precipitation.
   - *i.* Occurrence of wet and dry cycles in various localities. There are many long-term records of precipitation for localities scattered throughout the country. A study of synchronous occurrence of wet and dry cycles would be of value.
   - *j.* Development of more inexpensive types of automatic recording equipment, particularly rain gages. The development of some self-starting and self-stopping equipment for isolated stations would undoubtedly prove valuable.
2. Snow and Ice
   - *a.* Effect of temperature on rate of melting of snow.
   - *b.* Variation in water content of snow during the season; also its variation with depth of cover and temperature.
3. Evaporation
   - *a.* Evaporation from snow and ice.
   - *b.* Relationship between evaporation from large open water surfaces and from various sizes and kinds of land and floating evaporation pans. The experiments of the late Reuben B. Sleight, Assoc. M. Am. Soc. C.E., in Denver, and of Carl Rohwer, Assoc. M. Am. Soc. C.E., at Fort Collins, Colo., are quite complete, but similar work could well be carried on in other localities.
4. Temperature
   - *a.* Effect of various types of ground cover on temperature. This cover should include various grasses, pavements, soils, etc.
   - *b.* Relationship between temperatures on the ground and at Weather Bureau stations on tops of buildings, particularly on those heated during the winter. It might be possible to expose temperature-recording equipment at previous locations of Weather Bureau gages in order to reduce past temperature records to a common denominator.

COOPERATION OF COLLEGE DEANS ASKED

Many other lines of investigation would involve some outlay for meteorological apparatus. The committee would be pleased to hear from any institutions that happen to be well equipped in this particular. Inasmuch as the school year will be starting shortly, and senior and graduate students will be giving thought to their theses, the committee would appreciate hearing, at as early a date as possible, from heads of institutions who may contemplate such a program in order that research theses of a similar character in different sections of the country may be coordinated and standardized. More explicit facts will be given in individual cases.

COMMITTEE ON METEOROLOGICAL DATA
Donald M. Baker, *M. Am. Soc. C.E., Chairman*

108 West Sixth Street
Los Angeles, Calif.

Horatio Allen Scholarship

YOUNG ENGINEERS have shown much interest in the possibility of winning the Horatio Allen Scholarship at Columbia University during the coming year. After studying the qualifications of the many applicants, the Society's Columbia Scholarship Committee has recommended that the award be given to J. Gordon Lippincott, Jun. Am. Soc. C.E. This decision has received the endorsement of the Board of Direction, and in addition Mr. Lippincott has been accepted by the University.

It will be remembered that this scholarship is open to nomination by the Society each year. Including the allowances for fees, room accommodations, and books, it represents a total value of about $700. Very appropriately it is named for Horatio Allen, an illustrious engineer of the Class of 1823 at Columbia, who served as fifth President of the Society in 1872–1873.

It is believed that Mr. Lippincott will uphold the standards of the Society and of the scholarship. He was an Honor Man in Civil Engineering at Swarthmore College in 1931. Besides his academic accomplishments, he was president of the Student Chapter of the Society and prominent also in athletics, having been a member of the football team and captain of the track squad.

Since graduation, Mr. Lippincott has been in the employ of the Westchester County Park Commission and an assistant in the high school at White Plains, N.Y. His ambition is to specialize in the teaching of engineering. The committee on the Columbia University Scholarship consists of Robert Ridgway, Chairman, C. W. Hudson, and J. P. H. Perry, Members Am. Soc. C.E.

First Ballot on Society Nominees for 1933

OFFICIAL NOMINEES FOR VICE-PRESIDENTS AND DIRECTORS

THE PRELIMINARY or first ballot on official nominees for those Society offices that become vacant in January 1933, was canvassed on August 1, 1932. Shortly the second ballot for official nominees will be distributed, and this in turn will be canvassed on October 15. The final vote on the officers nominated will be taken during December and January, and the result will be announced at the Annual Meeting in 1933.

For purposes of information and record, the report of the tellers on the first ballot is printed here:

New York, N.Y.
August 1, 1932

To the Secretary
American Society of Civil Engineers:

The tellers appointed to canvass the first ballot for official nominees report as follows:

Total number of ballots received		1,957
Deduct:		
Ballots from members in arrears of dues	188	
Ballots without signatures	6	
Ballots from members who have resigned since voting	1	
Total not entitled to vote		195
Ballots canvassed		1,762

For Vice-President, Zone II

Frank O. Dufour	351
Frank L. Nicholson	198
Scattering	20
Total	569

For Vice-President, Zone III

Frank G. Jonah	245
James L. Ferebee	106
Scattering	96
Blank	2
Total	449

For Directors, District 1 (two to be elected)

J. P. H. Perry	254
James F. Sanborn	265
D. B. Steinman	17
Scattering	138
Blank	1
Total	336*

For Director, District 4

Henry J. Sherman	171
Scattering	9
Blank	23
Total	203

For Director, District 11

Ralph J. Reed	165
Scattering	19
Total	184

For Director, District 14

W. W. Horner	85
Scattering	15
Blank	6
Total	106

For Director, District 15

Edward N. Noyes	132
O. H. Koch	55
E. P. Arneson	25
Scattering	4
Total	216

Respectfully submitted,
LYNNE J. BEVAN, *Chairman*

Andrew Kolberk	Theo. Human, Jr.
Martin C. Knabe	J. Farhi
T. R. Galloway	R. J. Stark
H. J. Alexander	Halsey French
M. N. Quade	William L. Sylvester
Jacob Mechanic	G. R. Heckle
Ellis E. Paul	George E. Barnes
J. D. Moore	H. H. Pitcairn

L. M. Gray

Tellers

* *Number of valid ballots cast, on most of which were two nominations for Director.*

Lantern Lectures Have Useful Year

THE PRECEDING COLLEGE YEAR has witnessed an increasing use of the material in the form of lantern lectures, prepared for the use of Student Chapters. These lectures, sent without charge to any Chapter, have been the subject of discussion at a total of 230 meetings during the two semesters. The most popular titles were the "Coolidge Dam," which was given 22 times, and the "Holland Tunnel," a close second, with 21 hearings. Even the least popular of the series was asked for 14 times by different groups.

Three characteristic extracts, from reports of as many Chapter secretaries, show how this service is appreciated:

"These slides proved to be one of the most interesting features of the year which have been presented to the upperclassmen in the Civil and Mechanical Engineering Departments. The slides were shown twice to a total of about 175 juniors and seniors, and 8 professors."

"The group was composed entirely of students, and I believe that, with the exception of two or three, all were members of the Student Chapter.... Every one seemed exceedingly pleased with the slides and talk. The speaker had utilized many other sources of information, which added greatly to the presentation of the subject."

"We find that these lectures always draw a crowd. The one last night held an especial interest because of the very close relation between the lecture and the structural work being done by the seniors at the present time."

This material has occasionally been loaned to summer schools or to other interested groups besides the Student Chapters. Inquiries as to its use are welcomed at any time during the year.

Costa Rican Engineers Adopt Society's Code of Ethics

FROM the Facultad de Ingenieria de Costa Rica comes word that on June 1, 1932 it adopted for its official use the Code of Ethics of the Society. This fact is interesting not only for its own sake but because it evidences a feeling of unity among engineers in North America. Such action should go far toward establishing high standards for professional and ethical conduct, and broadening their application.

Among the officers of the Facultad de Ingenieria de Costa Rica recently elected are J. F. Salazar Quezada, Assoc. M. Am. Soc. C.E., president; and Ricardo Fernandez Peralta, secretary.

Public Speaking Contests for Juniors

DURING THE PAST spring, an interesting course of lectures under the sponsorship of the Metropolitan Section was given for the benefit of the Junior Members of the Society. This course was under the direction of Alexander Haring, M. Am. Soc. C.E. Recitations were made by the group of Juniors comprising the class. On June 14 the series culminated in a public speaking contest open to those who took the course. In all, nine contestants delivered speeches, after which prizes were awarded to the following members of the class:

James J. Knox, first prize, ten dollar gold piece, for his speech on the subject of "The Dole"; Alfred Wagner, second prize, a book, his subject being "Sensationalism in the Press"; Bernhard F. Bieman, Honorable Mention, for his speech on "What Makes, People Buy." The judges were William E. Reeves, William F. Shea, and Charles E. Trout, Members Am. Soc. C.E. The first prize was donated by Mr. Reeves, and the second by E. R. Needles, M. Am. Soc. C.E., members of the committee in charge.

The members of the class have expressed warm appreciation of this course, which was made available to them without cost by the Metropolitan Section. As an evidence of their interest and class spirit, they have formed a permanent organization with Clare N. Leonard as president and Neil R. McLeod as secretary. The purpose of this organization is to secure continuation of the work of this group, with two principal objects in mind; first, to extend their study and practice in public speaking; and, second, to be of service to the Metropolitan Section in such activities as they might properly participate in.

The plan now is to assign one Junior to membership on each of the standing committees of the Section. It is believed that real progress is being made through these activities in improving materially the status and future progress of the Juniors, both as individuals and in their relation to the Section, the Society, and the Profession.

Two Society Meetings

BY A COINCIDENCE, this issue is largely given over to the technical features of two Society Meetings. The articles comprise exclusively the records of the engineering sessions at the Yellowstone Convention in July. Also included is the official program of the forthcoming Atlantic City Meeting, October 5–8, 1932. This combination is rather unusual and comprises more than the customary attention to Society gatherings in any one number. The coincidence comes about because of the early date of the Fall Meeting.

Each meeting number as it develops seems better than its predecessors. This is true in the case of the Yellowstone Convention. A wide variety of topics were treated in a most interesting way. Fortunately, the written record here presented has permitted an unusually full account of the meeting, not only the descriptive, but the illustrative part. The favorable reports of the Convention will be readily appreciated by perusal of these articles. The subjects were most fitting, and the presentations were admirable.

To those who will feel cause for regret at not attending the Yellowstone Convention there will be some consolation in the possibilities presented in the forthcoming Fall Meeting at Atlantic City. All those who are familiar with this locality are unanimous in the opinion that it is at its best during early October. On its part, the Local Committee has been doing strenuous service in making all necessary preparations, as may be seen from a perusal of the program, presented in this issue for the first time.

Student Prize Winners

DURING the commencement season each year it is the stimulating practice of various Local Sections to present awards to engineering students graduating with high scholastic distinction from nearby colleges and universities. Subject to the approval of the Board of Direction, the recipient of one of these honorary awards is permitted to join the Society as a Junior, and his initiation fee and dues for the first year are paid by the Local Section. To date the following prize winners for 1932 have been reported to Society Headquarters:

NAME OF STUDENT	COLLEGE	LOCAL SECTION GIVING AWARD
William Taft Collins	Ohio State University	Central Ohio
Frederick Sorensen Cresswell	University of Cincinnati	Cincinnati
Harold Phillip Pickering	University of Colorado	Colorado
Lawrence Cousins Whitsit	University of Michigan	Detroit
Christian Elmer Carlsen	Purdue University	Illinois
Elmer Laurence Foster	Purdue University	Illinois
William Perry Jones, Jr.	University of Illinois	Illinois
Charles Udell Kring	University of Illinois	Illinois
Joe John Koperski	Lewis Institute	Illinois
Maynard Adams Patton	Northwestern University	Illinois
Torguy Joel Westerberg	Armour Institute of Technology	Illinois
Howard Leslie White	Rose Polytechnic Institute	Illinois
Homer Judkins Scott	Iowa State College	Iowa
Roland August Kampmeier	State University of Iowa	Iowa
Charles Elverton Ward	Cornell University	Ithaca
Paul Clutter Perry	Kansas State College	Kansas State
Charles Hottel French	Lafayette College	Lehigh Valley
Charles Courtney Seabrook	Lehigh University	Lehigh Valley
John Edward Rogan, Jr.	Tulane University	Louisiana
Maurice Carl Kipp	University of New Mexico	New Mexico
Gordon Ephraim Bodien	University of Minnesota	Northwestern
Milton Elmer Schmidt	University of Minnesota	Northwestern
Robert Kennedy Barclay	University of Pittsburgh	Pittsburgh
Thomas Robinson Cantine	Oregon State College	Portland (Ore.)
Robert Chalfin Smith	Oregon State College	Portland (Ore.)
Orville Kofoid	Oregon State College	Portland (Ore.)
Raymond Emile Gauthier	University of California	San Francisco
Carl Clifford Hallvik	University of Idaho	Spokane (Wash.)
Stifel William Jens	Washington University (St. Louis)	St. Louis
Carter Seddon Showell	University of Utah	Utah
Herbert Glemm Snellings	Virginia Polytechnic Institute	Virginia

Appointments of Society Representatives

C. W. KUTZ, M. Am. Soc. C.E., has accepted an appointment to serve on the Society's Committee on Meteorological Data.

J. VIPOND DAVIES, M. Am. Soc. C.E., has been appointed a member of the Hoover Medal Board of Award, to fill the vacancy caused by the death of WILLIAM BARCLAY PARSONS, Hon. M. Am. Soc. C.E.

T. H. UPTON, M. Am. Soc. C.E., served as Society representative at the official opening of the Sydney Harbor Bridge, Sydney, Australia, on March 23, 1932.

A Preview of Proceedings

Three main papers will form the chief content of the September issue of PROCEEDINGS. *The first deals with the analysis of the results of tests on full-sized built-up structural columns; the second will deal with the construction of the Martinez-Benicia Bridge across Suisun Bay, California, for the Southern Pacific Railroad; and the third, with the subject of prequalification of contractors, as presented at two separate meetings of the Highway Division in 1930, and now opened for discussion by the Society at large.*

TESTS OF RIVETED AND WELDED STEEL COLUMNS

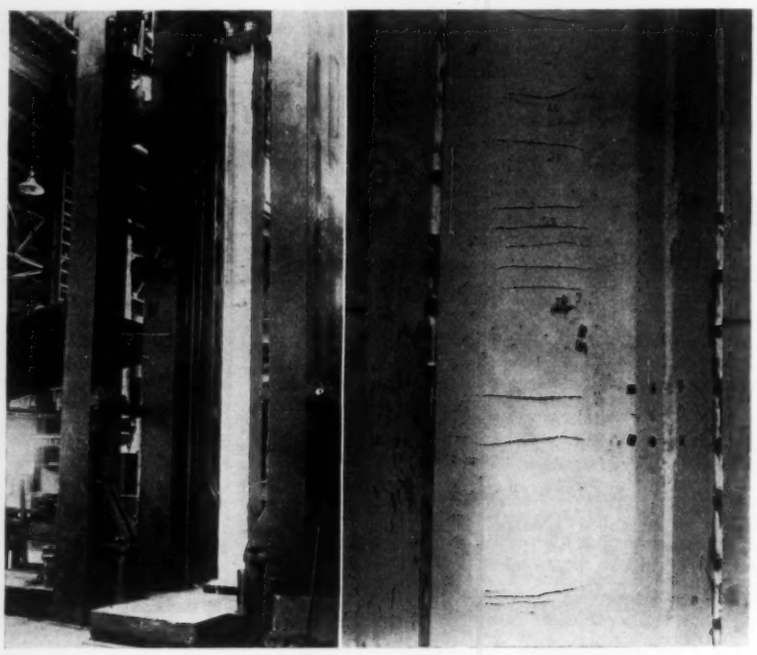
The tests described in the paper by M. O. Fuller and the late W. A. Slater, Members Am. Soc. C.E., permit a comparison of the behavior and strength of built-up steel columns fabricated by riveting with those of similar columns fabricated by welding. Nine columns were tested in the Fritz Engineering Laboratory at Lehigh University. Of these, two were riveted, three were welded intermittently, and four were welded continuously. The columns were built up by riveting or welding cover plates to the H- or I-sections that formed the core. All of them were tested as flat-ended columns.

The authors discuss the method of measuring strains, the slip of the plate, the deflection of the columns due to applied load, the strains caused by plastic flow of the heated metal, as well as the strains set up when the heated metal cools.

Before testing, each column was given a coat of white cement paint, so that, when the load was applied and the mill scale flaked off, the "strain lines" would be clearly visible. Discussion of these visible phenomena forms an important part of the paper.

The authors summarize their major results in nine conclusions: (1) The largest slippage of plate observed was about 0.001 in. (2) Stitch welding caused shortening of the metal at sections through the weld and elongation at the edges of the cover plate at sections midway between welds. (3) The first strain lines appeared at the points of highest stress caused by the heating of the metal, as nearly as could be determined. (4) In computing stress from strain, a mean observed value of 29,000,000 lb. per sq. in. was used as the modulus of elasticity. (5) Deflections were greatest for the continuously welded columns. (6) No marked advantage was indicated between riveted and welded columns so far as freedom from bending moments was concerned. (7) There was a general correspondence between the stress at the limit of proportionality and that at which strain lines first appeared, and this suggests that both were influenced by the presence of initial stresses in the columns. (8) The ratios of the stress at maximum load to the yield point stress determined from the coupons were nearly equal for all the columns tested. (9) The average stress at maximum load was about 15 per cent greater than the strength computed by Rankine's formula.

In an appendix, the authors have segregated a detailed account of the laboratory methods and technic and the manner in which the tests were set up. In its entirety, the paper should be of outstanding importance to those interested in research and practical design problems in the field of structural steel.



COLUMN TESTS CONDUCTED AT LEHIGH UNIVERSITY
Strain Lines Become Visible When the Mill Scale Flakes and Breaks Through the Coat of White Cement Paint

The late Professor Slater was always a generous contributor to the publications of the Society and to the technical literature of the engineering profession. This paper, accepted for publication in PROCEEDINGS before his death, is one of his last contributions. His co-author, Professor Fuller, has generously consented to take the responsibility of answering all discussion that ensues.

THE MARTINEZ-BENICIA BRIDGE

Experiences of the Southern Pacific Company in constructing a bridge across Suisun Bay in California are presented for discussion in the paper by W. H. Kirkbride, M. Am. Soc. C.E., to be published in the September issue of PROCEEDINGS. The paper represents a summarized record of engineering procedure on projects of this nature. The author discusses the geology of the site as it affected the design, and particularly the influence of two fault lines, the Southampton and the Green Valley. In addition to these two major fault lines there were numerous lines in a transverse direction. One of these was near enough to the bridge site to be a possible menace to the permanence of the structure. The question as to whether it passed under the bridge or not is discussed from the viewpoint of geologists and engineers.

A description of the bridge foundations and piers forms an important part of this paper. These include 2 U-abutments, 8 viaduct pedestal piers on land, and 14 viaduct pedestal piers and 10 truss-span piers in the water. Of the 10 piers in the water, 8 are classed as deep-sea piers, since they rest on a rock bottom at a depth of from 115 to 134 ft. below mean low water.

Of especial interest to members of our Construction Division is the fact that a deep-sea diver was employed in order to determine at all times the condition of the cutting edge of the caisson and the progress of the excavation. He noted particularly the relation of the cutting edge to bedrock and the quantity of excavation remaining to be done at any time. In all cases considerable clay or gravel was found compacted under the partition wall and banked up against the cutting edge.

By means of the information furnished by the diver the contractor was enabled to direct the jetting work until the foundation area was reported satisfactory and in a clean condition. The diver was able to supply accurate information as to the penetration of the cutting edge into the foundation rock and in each case he brought up samples of the rock encountered. According to Mr. Kirkbride, this was perhaps the first time a deep-sea diver has been used at such a great depth in open caisson work.

In his treatment of the superstructure, the author discusses the design from a theoretical point of view, gives primary dimensions and statistics, and describes the steps by which the various trusses and the lift spans were erected. The features worthy of special notice and discussion by the membership are listed by Mr. Kirkbride as follows: (1) intensive examination of earthquake faults and the design of bridge piers to resist earthquake shock; (2) the creation of artificial islands of sand within steel shells, by means of which the caisson bridge piers, entirely of concrete, were formed above water and sunk within the steel shell to bedrock; and (3) the temporary use of one of the permanent deck spans as falsework in the construction of the other bridge spans across deep water.

The Martinez-Benicia Bridge was successfully

MARTINEZ-BENICIA RAILROAD BRIDGE OVER SUISUN BAY, SAN FRANCISCO

placed in operation October 15, 1930. During the first 24 hours 39 passenger and freight trains crossed the bridge, and the lift span was opened four times. The structure was dedicated with fitting ceremonies on November 1, 1930.

PREQUALIFICATION OF CONTRACTORS

The principal topic adopted for discussion at the meetings of the Highway Division during the year 1930 was preliminary qualification of contractors, that is, qualification before they are allowed to submit bids. A meeting of the Division was held at Sacramento, Calif., in June 1930, in connection with the regular quarterly meeting of the Society, and another was held on October 2, 1930, in St. Louis, Mo.

Papers were presented by L. I. Hewes, M. Am. Soc. C.E.; by C. H. Purcell and W. P. Christie, Associate Members Am. Soc. C.E.; and by Walter Wilkinson, R. B. Brooks, A. J. Parrish, and W. W. Bass, and were discussed orally at the meetings at which they were presented. In this way the Highway Division has gathered together the opinions of representative highway engineers and administrators, general contractors, city administrators, and municipal engineers. The purpose of the paper by C. J. Tilden, M. Am. Soc. C.E., Strathcona Professor of Engineering Mechanics at Yale University, on "Prequalification of Contractors," which will appear in the September issue of PROCEEDINGS, is to present a concise, unbiased analysis of the material on prequalification of contractors thus assembled and to offer it to the Society at large for extended discussion. Prequalification of contractors is defined by Professor Tilden as "the determination of responsibility before plans and proposals are issued for bidding." The paper presents the legal aspects, the various viewpoints of contractor and owner, advantages and disadvantages of prequalification, and a summary of conclusions. Some form of qualification is recommended, but according to Professor Tilden it should not be so rigid or exacting as to prevent new contractors from entering the field.

News of Local Sections

ITHACA SECTION

The Ithaca Section had a dinner meeting at the Kalurah Country Club, Binghamton, N.Y., on Tuesday evening, May 24. After the dinner, A. Burton Cohen delivered a very interesting illustrated lecture on the subject of "Modern Tendencies in the Design of Concrete Structures." There were 31 members and guests present.

PITTSBURGH SECTION

Election of officers, held at the June 7 meeting of this Section, resulted as follows: P. J. Reich, President; E. N. Hunting, Vice-President; and Nathan Schein, Secretary-Treasurer.

SACRAMENTO SECTION

Recent meetings held by this Section have been well attended. Among the informative subjects discussed were: "Highway Rights-of-Way," by Frank B. Durkee, General Right-of-Way Agent for the State Division of Highways; "The Problems of a City Administrator," by A. J. Cleary, Chief Administrator of the City of San Francisco; and "Modern Development of Railroads," by W. F. Turner, Division Engineer of the Southern Pacific Railroad.

SPOKANE SECTION

At the regular monthly meeting of the Spokane Section, held on Friday, June 8, John W. Finch, Dean of the School of Mines of the University of Idaho, gave a very interesting talk on his trip to inspect the placer mining operations which he has been active in sponsoring. There are now between 18,000 and 20,000 men and women working the many streams of the Pacific Coast region.

Student Chapter News

UNIVERSITY OF GEORGIA

A wide range of engineering subjects was discussed at meetings held by the University of Georgia Student Chapter last year. Among these were: "Sand Clay Roads" and "Test Roads of Georgia." Pictures shown on various occasions included: "The Manufacturing of Iron," "The Development of Hydro-Electric Power," and the "Cascade Tunnel."

UNIVERSITY OF ILLINOIS

There were 11 meetings held by this Chapter during the past year. At a meeting held on May 25, Francis Lee Stuart, Past-President of the Society, and George T. Seabury, Secretary, were guest speakers. At other meetings, the following well known engineers were among those who spoke: D. B. Steinman, Ralph W. Street, Maj.-Gen. Lytle Brown, Professor Westergaard, Dr. Carol

Aronovici, and Maj.-Gen. T. Q. Ashburn. Other activities of the chapter included a "smoker" held on September 25, a "freshman mixer" on November 17, and an annual banquet.

UNIVERSITY OF WASHINGTON

During the past year the Student Chapter of the University of Washington has enjoyed a number of interesting meetings, at one of which Professor Harrison reviewed "Men and Machines." Many of the students went on the field trip taken to the Cle Elum Dam, a project of the U.S. Reclamation Service. This trip proved to be of great value and interest.

American Engineering Council

National representative of 26 engineering societies, with a constituent membership of 60,000 professional engineers, reports civil engineering news of the Federal Government

HOME LOAN BANK BILL PLACED IN OPERATION

PRESIDENT HOOVER, July 22, 1932 signed the bill, H.R.12280, "An act to create Federal Home Loan Banks, to provide for the supervision thereof, and for other purposes," and made it Public Law 304 of the 72d Congress. This measure was recommended by American Engineering Council's Committee on Government Expenditures. The measure is looked upon as a most constructive one, and an organization is now being formed to put the provisions of the act into operation. It is believed that before October, the operation of this law will relieve the pressure of mortgage foreclosures on homes and farms. The engineering profession was not only interested in this measure as one that offered hope and relief in a bad credit situation, but it advocated the measure as a possible aid in stimulating the construction industry in building homes during the coming years.

The law does not authorize an individual to obtain a loan directly from the Federal Government, but it does provide for the establishment of from 8 to 12 District Federal Home Loan Banks from which banks, building and loan associations, and other institutions may receive loans upon adequate security in the form of first mortgages.

ST. LAWRENCE DEEP SEA WATERWAY TREATY

Public announcement has been made of the terms of the treaty proposed with Canada relative to the development of the Great Lakes-St. Lawrence Deep Sea Waterway. The ultimate establishment of such a deep sea waterway has been approved and advocated in principle by American Engineering Council for a number of years.

Under the terms of the treaty, copies of which may be had upon application to the State Department, Washington, D.C., no actual construction work will be done on this project until the treaty is ratified by the U.S. Senate and the Parliament of Canada. This, of necessity, will require many months. The U.S. Senate does not meet again until next December and then it is expected, because of certain controversial features in the treaty, that it will be discussed for several months. The Senate has appointed a special investigating committee to study the treaty during the summer and fall months, which may bring about more rapid action, but it would be most unusual should this treaty be consummated within one year.

Article 3 of the treaty provides for a joint commission to handle all of the details of construction in the international section of the St. Lawrence River. This commission will be composed of five Canadians and five Americans. The American representatives will be appointed by the President of the United States, but cannot be appointed until the treaty is ratified by both countries.

EMERGENCY RELIEF AND CONSTRUCTION ACT OF 1932

The recent session of Congress passed several measures designed to stimulate considerably the undertaking of self-sustaining engineering projects and to give additional employment. The most notable of these acts was the Emergency Relief and Construction Act of 1932. This law has three major provisions. The first provides that states may borrow from the Reconstruction Finance Corporation at 3 per cent interest up to $300,000,000 for direct relief of suffering in the states. The second provides for loans to states and municipalities for self-liquidating projects. The third and final provision is for a possible increase in the public works construction program of the Federal Government.

The nature of this authorized work and the name and address of the Government official under whose jurisdiction it will fall, are given here:

1. Emergency highway construction, $120,000,000
   Highway Commissioners
   of the various states
2. Roads, trails, bridges, fire lanes, etc., in National Forests, etc., $16,000,000 divided as follows:
   a. National Forest highways, $5,000,000
   R. Y. Stuart
   Atlantic Building
   Washington, D.C.
   b. Construction and maintenance of roads, trails, bridges, fire lanes, etc., $5,000,000
   R. Y. Stuart
   Atlantic Building
   Washington, D.C.
   c. Roads and trails—National Parks, $3,000,000
   Horace M. Albright
   Director, National Park Service
   Interior Department Building
   Washington, D.C.
   d. Indian Reservation roads, $1,000,000
   Charles J. Rhoads, Commissioner
   Bureau of Indian Affairs
   Interior Department Building
   Washington, D.C.
   e. Public land roads, $2,000,000
   Charles C. Moore, Commissioner
   General Land Office
   Interior Department Building
   Washington, D.C.
3. Prosecution of river and harbor projects, $30,000,000
   Division or District Engineers
   of the Corps of Engineers, U.S.A.
4. Flood control projects, $15,500,000
   District or Division
   Engineers Officers, U.S.A.
5. Air navigation facilities, $500,000
   Col. Clarence M. Young
   Assistant Secretary for Aeronautics
   Department of Commerce
   Washington, D.C.
6. Construction and equipment, lighthouse tenders and vessels, $2,860,000
   George R. Putnam, Commissioner
   Bureau of Lighthouses
   Washington, D.C.
7. Engineering work, U.S. Coast and Geodetic Survey, $1,250,000
   Capt. R. S. Patton, Director
   U.S. Coast and Geodetic Survey
   Washington, D.C.
8. Bureau of Yards and Docks, Navy, $10,000,000
   Rear Admiral A. L. Parsons
   Chief of Civil Engineers of the Navy
   Navy Department, Washington, D.C.
9. Building construction outside the District of Columbia, public buildings, different kinds, $100,000,000
   Ferry K. Heath
   Assistant Secretary of the Treasury
   Washington, D.C.
10. Construction installation at military posts of buildings, utilities, and appurtenances, $15,164,000
    Maj. Gen. J. L. DeWitt
    Quartermaster General, Munitions Building
    Washington, D.C. (Or Commander of the Post where the construction is to be done)

ITEMS OF INTEREST

Engineering Events in Brief

Civil Engineering for October

WHEREVER engineers are engaged in the design, construction, or operation of subway systems, they will find the experience gained in New York valuable. In an article to appear in the October issue of CIVIL ENGINEERING, A. I. Raisman, M. Am. Soc. C.E., will explain the factors which have influenced both the general and the detailed arrangement of the stations on the New York subway systems.

A second article will describe the straight gravity overflow dam built at Graaff Reinet in South Africa to store water for irrigation. In January 1932 the reservoir was filled for the first time and a major flood was passed safely over the dam. The resulting erosion below the structure and the effect of the unusual stepped construction of the downstream face of the dam will be discussed in the article by Philip R. R. Bisschop, Assoc. M. Am. Soc. C.E.

Any advance information as to the probable run-off from a watershed is always of great value in making possible the most complete utilization of the water. The work of the state of Utah in forecasting run-off from snow measurements along selected courses in that state, and especially the methods and equipment used for the work, will be described by George D. Clyde, Assoc. M. Am. Soc. C.E., in another article.

The transportation industry, next to agriculture, is the largest of our national industries. Present-day problems of the railroads, the buses, the truck lines, the inland and intracostal waterways, and the highway systems are pointed out and discussed by J. S. Worley, M. Am. Soc. C.E., in the October issue. His long study of these problems will lend weight to his statements. In addition, W. W. Zass, M. Am. Soc. C.E., will analyze the primary principles of highway design from the standpoint of the economics of motor car operation. His data are presented in an interesting narrative form.

Other articles available for the October issue deal with the subjects of simplified design of concrete beams, the strengthening and reconstruction of an old railroad bridge, and the regulation of building sizes and heights in relation to street widths in congested urban areas.

Water Works as Self-Liquidating Improvements

THE AMERICAN Water Works Association is authority for the statement that in nearly every city water works betterments are needed to provide higher pressures in certain parts of the system, thus ensuring a better supply of water for domestic use and more effective fire protection. Researches also show that in many communities the water supply is inadequate both as to quantity and quality, so that new or additional supplies and purification works are needed.

In the opinion of the Association, water works properties are outstanding examples of self-liquidating public works, and for that reason the right to borrow money for the construction of water supply facilities has been extended to private corporations owning and operating water works. It is expected that all needed water works betterments and extensions will be carried forward immediately with the money available through the Reconstruction Finance Corporation, thus providing employment and stimulating trade recovery to the full extent intended by the Emergency Relief and Construction Act.

It is well known that construction costs are at present very low, so that good value is returned for every dollar spent. Water works betterment projects, like many similar engineering works, create employment for many men, not only on the construction contracts but also back at the mines, mills, and shops from which come the materials and equipment for the work.

These efforts of the American Water Works Association are in line with the action previously taken by the Board of Direction, sympathetic to the encouragement of the construction of deferred and self-liquidating water works projects.

Mansfield Merriman Portrait

LEHIGH UNIVERSITY has recently received a gratifying accession to its notable gallery in the form of a portrait of the late Mansfield Merriman, M. Am. Soc. C.E. Dr. Merriman was Lehigh's first Professor of Civil Engineering, serving continuously in this position from 1878 to 1907. He was even more famous as the author of a large number of widely used textbooks of engineering. The portrait is the gift of the members of the Class of 1897 of Lehigh University.

Another Engineer President

THE FOURTEENTH President of France, Albert Lebrun, elected to succeed Paul Doumer, recently assassinated, was educated as a mining engineer and followed this profession with notable success. According to press information from Paris, M. Lebrun entered the École Polytechnique at the age of 19 and was graduated first in his class. He then attended the École Nationale Supérieure des Mines before beginning the practice of his profession. During the few years he devoted to engineering, M. Lebrun received a prize from the Academy of Sciences and made himself known through a number of treatises on mining and railway questions.

He began his political career in 1900, when he was elected to the Chamber of Deputies. In 1920 he was elected to the Senate, of which he became president in 1931. It was from this post that he was advanced to the highest political honor in the power of the French Republic to give.

Engineers' Advisory Board to Pass on Self-Liquidating Projects

UNDER DATE of August 11, 1932, President Hoover announced the personnel of an Engineers' Advisory Board which is to pass on the eligibility of construction projects, as provided under the Emergency Relief and Construction Act of 1932. This is the committee referred to in the act as the Control Project Board. As announced, the board is to consist of the following well known engineers: Charles D. Marx, chairman; Professor Emeritus of Civil Engineering, Leland Stanford University, Palo Alto, Calif.; Maj.-Gen. Lytle Brown, Chief of Engineers, U. S. Army; John F. Coleman, New Orleans, La.; John H. Gregory, Professor of Civil and Sanitary Engineering, Johns Hopkins University, Baltimore, Md.; and John L. Harrington, consulting engineer, Kansas City, Mo.

It will be noted that every one of these men is a Member of the Society. In addition, the chairman is a Past-President and Honorary Member; Mr. Coleman is also a Past-President; Mr. Gregory is a member of the Board of Direction; and Mr. Harrington is a former president of the American Society of Mechanical Engineers.

It is understood that as yet the routine, procedure, and machinery of the board are not entirely developed, although it is expected that they will be as quickly as possible, so that the board can deal promptly with projects put before it. Headquarters have been established at 1835 H Street, N.W., Washington, D.C. The members of the board are devoting their full time to this important work.

NEWS OF ENGINEERS

From Correspondence and Society Files

JOHN R. NICHOLS, consulting engineer, and CHARLES E. NICHOLS, formerly with Stone and Webster Engineering Corporation, are now associated as consulting structural engineers with offices at 45 Newbury Street, Boston.

GEORGE B. RUMBLE is now Assistant Engineer with Albright and Friel, Inc., with headquarters in Philadelphia.

R. G. TABER has accepted a position with the Fall River Gas Works Company as vice-president and general manager. Formerly, Mr. Taber was Superintendent of Construction for the Stone and Webster Engineering Corporation.

GEORGE KINGSLEY has taken a position with the U.S. Sugar Corporation as Superintendent of Water Works.

KENNETH S. LANE has accepted a position as Field Engineer with the A. L. Hartridge Company, of New York City.

DON M. FORESTER is now connected with the U.S. Bureau of Reclamation with headquarters in Boulder City, Nev. Previously Mr. Forester was Chief Engineer for the Seabrook Engineering Corporation, of Moscow, Union of Socialistic Soviet Republics.

JOHN DOUGLASS FITCH has joined the staff of American Engineering Council, as engineering associate. He was formerly with the Charles B. Hawley Engineering Corporation in Washington, D.C.

RUSSELL C. BOOTH, formerly employed by the Electric Bond and Share Company of New York City, is at present associated with the Callahan Construction Company as Field Superintendent on Madden Dam.

VIRGIL L. MINEAR is connected with the U.S. Bureau of Reclamation, located at Boulder City, Nev.

J. RHODES SMITH has accepted a position with Walter Kidde Constructors, Inc., of New York City, as Designing Engineer. Previously Mr. Smith was Engineer of Structures for J. E. R. Carpenter, also of New York City.

E. A. RUDULPH has entered the employ of the Union Electric Light and Power Company of St. Louis, after severing his connection with the Stone and Webster Engineering Corporation of Eldon, Mo.

JOE WALLACE KELLY, at present employed by the Engineering Materials Laboratory of the University of California, was previously in the Engineering Extension Department of Purdue University.

A. C. LINGLEY, until recently sales manager for the Carter-Waters Corporation of Des Moines, Iowa, is now with the Wheeler Lumber Bridge and Supply Company in Kansas City, Mo.

C. E. GRIGGS has been appointed City Engineer and Superintendent of Streets of Phoenix, Ariz.

GEORGE R. CLEMENS has become Senior Engineer with the Mississippi River Commission, Vicksburg, Miss. He was previously an engineer in the U.S. Engineer Office, Vicksburg.

Changes in Membership Grades

Additions, Transfers, Reinstatements, Deaths, and Resignations

From July 9 to August 9, 1932

ADDITIONS TO MEMBERSHIP

ALLISON, WILLIAM NOEL (Jun. '32), 5264 Nineteenth Ave., N.E., Seattle, Wash.

CORNICK, FREDERICK JOSEPH (Assoc. M. '32), Asst. Engr. and Hydrographer, Los Angeles County Flood Control Dist., Los Angeles (Res., 344 South Santa Anita Ave., Pasadena), Calif.

CRAVITZ, PHILIP (Jun. '32), 106 South Cummings St., Los Angeles, Calif.

DAIL, WILLIAM ELTON (Jun. Oct. '31), 1426 East 5th St., Tucson, Ariz.

ECKERLE, WILLIAM PETER (Assoc. M. '32), Asst. Bridge Engr., State Highway Dept. (Res., 326 West Main St.), Frankfort, Ky.

FENERDJIAN, HRANT ARMENAG (Jun. '32), Care, Societé Anonyme Hellenique des Eaux, Stadium Str. 4 (Res., Lianghi 6 A), Athens, Greece.

GAUCKLER, ANTHONY JOSEPH (Assoc. M. '32), Structural Engr., Milwaukee School Board (Res., 2232 North 29th St.), Milwaukee, Wis.

GIBSON, JOHN BRUCE (Jun. '32), 1933 Casa Grande St., Pasadena, Calif.

HENRY, MAXWELL (Assoc. M. '32), Instr. in Eng., School of Technology, Coll. of the City of New York, 139th St. and Convent Ave., New York, N.Y.

HIPP, CHARLES LEO (Jun. '32), Draftsman, U.S. Engrs. (Res., 3616 Olive St.), Kansas City, Mo.

HIPP, THOMAS MARTIN (Assoc. M. '32), With Phillips Petroleum Co., Box 268 (Res., 3616 Olive St.), Kansas City, Mo.

HOLLIS, ROBERT WOOD, JR. (Jun. '32), Senior Eng. Office Aid, Div. of Highways, State Dept. of Public Works, State Bldg., San Francisco (Res., 2423 Blake St., Berkeley), Calif.

JOHNSON, IRVING LAURENCE (Jun. '32), Res. Engr. and Designer, County of Sacramento (Res., 2643 Portola Way), Sacramento, Calif.

KILLMER, ROBERT EDWARD (Assoc. M. '32), Res. Engr., Texas State Highway Dept., Refugio, Tex.

MCKINNEY, GERALD FRANCIS (Assoc. M. '32), Sales Engr., The Peebles Paving Brick Co. (Res., 1022 Kinney's Lane), Portsmouth, Ohio.

MARTIN, GEORGE HARRIS, JR. (Assoc. M. '32), Partner and Mgr., of Harrison Office, Chas. H. Sells, Inc., Pleasantville (Res., 16 Hyatt Ave., Harrison), N.Y.

MINAMI, JOHN KAZUO (July '32), 301-C Holden Green, Cambridge, Mass.

NICHOLS, GEORGE HENRY (Jun. '32), Draftsman, B. M. Hall & Sons, 701 Peters Bldg. (Res., 768 Cascade Ave., S.W.), Atlanta, Ga.

O'DONNELL, JAMES PATRICK (Jun. '32), 117-34 One Hundred Fortieth St., South Ozone Park, N.Y.

OLAFSEN, REIDAR (Assoc. M. '32), Structural Designer, New Kanawha Power Co., Glen Ferris, W.Va.

ORCUTT, FORREST GLENN (Assoc. M. '32), Junior Engr., Bureau of Public Roads, 237 Custom House Bldg. (Res., 1825 South Emerson St.), Denver, Colo.

PAULUS, RAYMOND LAWRENCE (Assoc. M. '32), Engr., W.C. Babcock Grain Co., Rensselaer, Ind.

PEARSON, WALTER HERBERT (Assoc. M. '32), Plant Engr., Bituminous Service Co., West Chester (Res., 228 Lincoln Ave., Manoa), Pa.

SANDBERG, CLIFFORD HELMER (Assoc. M. '32), Asst. Engr., Bridge Dept., A.T. & S.F.R.R. (Res., 7752 Eastlake Terrace), Chicago, Ill.

SONNE, JULIUS ASA (Jun. '32), 1109 Capuchino Ave., Burlingame, Calif.

STONE, HENRY NORTON (Assoc. M. '32), Asst. Erection Engr., Virginia Bridge & Iron Co., Birmingham, Ala.

TAYLOR, DAVID YEATON (Jun. '32), Transitman, City of Medford, Medford (Res., 22 Usher Rd., West Medford), Mass.

MEMBERSHIP TRANSFERS

ANDREWS, ERIC ALEXIS (Jun. '28; Assoc. M. '32), Engr. of Sewers, Dept. of Public Works (Res., 85 Rockledge Ave.), White Plains, N.Y.

ANGILLY, CHARLES ENOCH, JR. (Assoc. M. '27; M. '32), Engr. of Design, Dept. of Water and Power, 207 South Broadway (Res., 1152 West 74th St.), Los Angeles, Calif.

AVERS, WILLIAM DEWITT (Jun. Oct. '26; Assoc. M. '32), Mgr., Islip Office, Wallace H. Halsey, Inc., Box 30, Islip, N.Y.

BELL, HOWARD FRED (Jun. '10; Assoc. M. '16; M. '32), Civ. Engr. and County Surv., Park County, Cody, Wyo.

BROWN, ALBERT ABRAHAM (Jun. '27; Assoc. M. '32), Structural Engr., H. H. Robertson Co., 2000 Grant Bldg. (Res., 5735 Hobart St., Apartment 1-B), Pittsburgh, Pa.

CORSON, HAROLD HAWLEY (Assoc. M. '28; M. '32), Village Engr., Municipal Bldg. (Res., 336 West Maple), Birmingham, Mich.

FREEMAN, CASSIUS WILLIAM (Jun. '27; Assoc. M. '32), With Metropolitan Dist. Water Bureau, Hartford (Res., 336 Brewer St., East Hartford), Conn.

GAYTON, LORAN DE LANCY (Assoc. M. '21; M. '32), Asst. City Engr., 402 City Hall, Chicago, Ill.

GRIFFENHAGEN, EDWIN OSCAR (Assoc. M. '11; M. '32), (Griffenhagen & Associates) 221 North La Salle St., Suite 3916 (Res., 56 East Elm St.), Chicago, Ill.

HALL, BENJAMIN MORTIMER, JR. (Assoc. M. '17; M. '32), Civ. and Min. Engr. (B. M. Hall & Sons), 701 Peters Bldg., Atlanta, Ga.

KAESTNER, ALBERT CARL (Jun. '12; Assoc. M. '16; M. '32), Secy-Treas. and Director, Aqua Systems, Inc., 2 Lafayette St., New York, N.Y.

LINEBERGER, WALTER FRANKLIN (Assoc. M. '09; M. '32), 3651 Pacific Ave., Long Beach, Calif.

NEWSOM, CLIFFORD COOK (Assoc. M. '28; M. 32), Dist. Engr., State Highway Comm., Box 2, Crawfordsville, Ind.

PARKS, WARREN WRIGHT (Assoc. M. '22; M. '32), Res. Engr., Thomas J. Emery Memorial, 3800 Plainville Rd., Mariemont, Cincinnati, Ohio.

REAM, WILBUR BARNER (Assoc. M. '29; M. '32), Designing Engr., Middle Rio Grande Conservancy Dist., Court House (Res., 1205 Las Lomas Rd.), Albuquerque, N.M.

SHUBIN, SYDNEY ABRAM (Assoc. M. '26; M. '32), Asst. Design Engr., Bureau of Bridges, Dept. of Public Works, Allegheny County (Res., 998 Biltmore Ave., Dormont), Pittsburgh, Pa.

SLASON, EARLE BERNICE (Jun. '29; Assoc. M. '32), 417 Grand Ave., Kansas City, Mo.

SOULE, JOHN EDWARD (Jun. '24; Assoc. M. '32), Pres. and Treas., Soule Contr. Co., Box 1504, Pensacola, Fla.

WARREN, DONALD REXFORD (Assoc. M. '28; M. '32), Engr., County of Sacramento (Res., 1416 Fifty-Second St.), Sacramento, Calif.

RESIGNATIONS

BELL, HAROLD SILL, Assoc. M., resigned July 28, '32.

CERNY, JOHN WILLIAM, Assoc. M., resigned July 19, '32.

EEPTING, THEODORE, Jun., resigned July 25, '32.

GALE, ALBERT GARLAND, Assoc. M., resigned July 18, '32.

GASS, HOWARD RAY, Assoc. M., resigned July 15, '32.

GREENE, RUSSELL DE COSTA, M., resigned July 15, '32.

HANSON, LEROY EDMUND, Jun., resigned Aug. 3, '32.

HARRIS, GEORGE HENRY, Assoc. M., resigned July 11, '32.

HAYDEN, BEAUFORD EMMETT, M., resigned July 12, '32.

IDLE, LEONARD AUGUSTUS, Assoc. M., resigned July 12, '32.

MIKELL, WARING, Jun., resigned July 12, '32.

TELLER, WILBUR WINFIELD, Jun., resigned July 22, '32.

UNDERWOOD, WALTER CHARLES, Jun., resigned July 28, '32.

WARD, WILLIAM BERNARD, Assoc. M., resigned July 9, '32.

DEATHS

ALISON, THOMAS HENRY. Elected M., Oct. 7, 1908; died July 2, 1932.

GRESHAM, ROBERT HALL. Elected M., May 4, 1898; died July 19, 1932.

HUNT, CHARLES WARREN. Elected M., Jan. 2, 1884; died July 23, 1932.

LARKINS, EDGAR ERNEST. Assoc. M., Sept. 6, 1910; M., Jan. 16, 1928; died May 18, 1932.

MCLEAN, JOHN ALEXANDER. Elected Assoc. M., April 3, 1922; died July 30, 1932.

OLNEY, ALFRED CLARENCE. Elected Jun., April 30, 1895; M., Nov. 2, 1898; date of death unknown.

WHEELER, WILLIAM. Elected M., Oct. 4, 1893; died July 1, 1932.

TOTAL MEMBERSHIP AS OF AUGUST 9, 1932

Members	5,866
Associate Members	6,383
Corporate Members	12,249
Honorary Members	17
Juniors	2,963
Affiliates	122
Fellows	5
Total	15,356

Men Available

These items are from information furnished by the Engineering Societies Employment Service with offices in Chicago, New York, and San Francisco. The Service is available to all members of the contributing societies. A complete statement of the procedure, the location of offices, and the fees is to be found on page 97 of the 1932 Year Book of the Society. Unless otherwise noted, replies should be addressed to the key number, Engineering Societies Employment Service, 31 West 39th Street, New York, N.Y.

CONSTRUCTION

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 32; married; graduate; 8 years with large construction company on reinforced concrete and finished type structures; accounting, cost, testing, subcontracts, and superintendent; 5 years renting and maintaining real estate while at college. Desires connection in construction or allied fields or real estate. B-8171.

CONSTRUCTION ENGINEER; Assoc. M. Am. Soc. C.E.; 56; married; graduate of Massachusetts Institute of Technology; 16 years field and office engineering—railroads and gas and electric company; 4 years engineer and manager of branch office for large contracting company; 13 years municipal and state engineering. Available immediately. Prefer location in New England states. D-1254.

ENGINEER; Assoc. M. Am. Soc. C.E.; 32; graduate; 5 years responsible positions with contractors; 5 years in municipal and highway engineering, construction, and studies; 1½ years on technical committee for the Inter-American Highway Surveys, Central America. Speaks and writes French and Spanish. Desires connection with contractors or manufacturer. D-1367.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 39; married; 19 years varied engineering experience in field and office. Design and supervision of construction and as contractor, on subways, elevated railroads, highways, bridges, municipal improvements, subdivisions, and reinforced concrete structures. Available immediately. Location anywhere, preferably Pacific Coast. D-1301-327-A-2 San Francisco.

FIELD ENGINEER; Jun. Am. Soc. C.E.; 28; married; 2 years experience tunnel construction; 4 years building construction; 2½ years railroad surveying and trackwork layout. Good field man for contractor, architect, or utility company on any type of construction, or in any capacity. Small salary to start if position is permanent. C-5895.

GRADUATE CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 8 years varied structural experience in drafting, design, and construction; estimates, inspection, surveys, layouts, and related work. Readily adaptable to any position. Desires connection with construction company, consultant, contractor, or architect, in field or office; also sales-engineering or instructorship. C-2605.

CONSTRUCTION ENGINEER; Assoc. M. Am. Soc. C.E.; 42; married; North and South Carolina registration; 20 years experience on highways, sanitary sewers, water supply, paving, and industrial plants; 3 years in charge of topographic and property surveys; 6 years on hydro-electric construction; 7 years as resident engineer in charge of large projects. Available immediately. Southern location preferred. D-1379.

GRADUATE CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; member, New York State Society of Professional Engineers; licensed professional engineer and land surveyor, New York State; 6 years responsible structural design and resident engineer on construction of sewage treatment plants. Connection with contractor or consulting engineer preferred. C-2907.

DESIGN

CIVIL AND STRUCTURAL ENGINEER; Jun. Am. Soc. C.E.; 28; B.S. in C.E. and C.E. degrees; about 5 years experience in New York City as a designer, detailer, draftsman, checker, and estimator of steel and reinforced concrete. References and samples of work sent on request. C-6186.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 34; married; graduate of Rensselaer Polytechnic Institute; 1 years experience in design of industrial buildings; 8 years in construction of caissons, tunnels, subways, and buildings; 4 years in design of air-conditioning systems and apparatus. Available immediately. D-1260.

GRADUATE CIVIL ENGINEER; Jun. Am. Soc. C.E.; electric arc welder; experience in design of welded structures; 2 years surveying; 3 years production. Desires work in field of welding engineering. Available immediately. D-1324.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; S.M. in engineering; B.S. in C.E.; New York State licenses; experienced in design of indeterminate structures; has taught mathematics; 2 years in charge of design, plans, and specifications for buildings, bridges, water supply, and drainage systems. Total experience, including college, 11 years. D-1333.

STRUCTURAL ENGINEER; Assoc. M. Am. Soc. C.E.; 34; married; university graduate; 6 years experience designing railroad and highway bridges of all types, steel and concrete arches, etc. (4 years in responsible charge); 2½ years designing office and mill buildings, steel and concrete, in responsible charge; 1 year field work (bridges and buildings). D-1369.

STRUCTURAL ENGINEER; Assoc. M. Am. Soc. C.E.; 30; university graduate; 2 years structural detailing and checking; 2 years bridge designing—railroad and highway bridges; 2 years in responsible charge of structural designing in connection with railroad electrification; 1½ years in responsible charge of design of office and industrial buildings. One month's free service for trial. C-9066.

STRUCTURAL ENGINEER; Assoc. M. Am. Soc. C.E.; 39; married; graduate C.E.; 18 years experience in surveys, design, and details of steel buildings, bridges, reinforced concrete, plant layout, elevators, conveyors, transmission, etc. Available immediately. D-1028.

EXECUTIVE

ENGINEER-EXECUTIVE; M. Am. Soc. C.E.; 52; graduate in law and mechanical and civil engineering; M.S. in municipal and highway engineering; 18 years as executive in investigation, design, reports, electric railway and power plant operation, highway, railway, bridge, building, drainage, and flood control construction. Fluent Spanish. Desires position in Southwest or Spanish-speaking countries. B-6357.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 43; married; graduate civil engineer; 20 years experience; New Jersey license; last 10 years as field engineer with Portland Cement Association; contact with contractors, state, county, and municipal officials; design and construction of concrete pavements. Desires connection in cement industry. Location immaterial, if position is permanent. Available immediately. D-1250.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 21 years experience—industrial construction, rapid transit (legal engineering), city development, and purchasing. Desires to represent American firm in Canada or England. D-932.

STRUCTURAL ENGINEER; Assoc. M. Am. Soc C.E.; 31; married; 10 years intensive experience with prominent consulting engineers estimating, designing, investigating, and preparing drawings for substructures and superstructures of fixed bridges and the structural and mechanical features of movable bridges. Accustomed to responsibility. References from former employers and their clients. D-1258.

STRUCTURAL ENGINEER; Assoc. M. Am. Soc. C.E.; graduate M.E. and C.E.; licensed; 21 years experience. Last 9 years in structural steel and concrete buildings and bridges. Design, estimating, shop detailing. Able to lead men and accept full charge. Stable position preferred. Foreign service considered. Best references. B-8303.

CIVIL AND STRUCTURAL ENGINEER; Assoc. M. Am. Soc. C.E.; married; licensed professional engineer, state of New York; 25 years experience in design and construction of hydro-electric and steam power plants, transmission lines, industrial plants, electric railways, estimates, specifications, and purchasing. Desires responsible charge of work. Location, New York. B-5423.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 35; licensed; 17 years experience in construction, layout, and design of industrial plants, crushing plants, etc., including furnaces, heavy machinery, structural steel, and foundations. Can take full charge of drafting room and get work out economically. Immediately available, any location. B-5715.

INDUSTRIAL ENGINEER AND ARCHITECT; M. Am. Soc. C.E.; graduate; licensed; 20 years experience in design and construction of chemical and other industrial plants, hydro-electric developments, housing groups, commercial garages, and all water and sewer systems, and other utilities in connection therewith. Organization and supervision of office and field forces. Preliminary investigations and reports. B-2835.

ENGINEER EXECUTIVE; Assoc. M. Am. Soc. C.E.; graduate C.E., 1914; excellent record; 12 years design; 6 years field. Heavy foundations, tunnels, railway terminals, viaducts, power house, and miscellaneous industrial plant. Competent engineering business, organizing work and forces. Will consider any assignment. C-1445.

CONSTRUCTION EXECUTIVE AND ARCHITECT; Assoc. M. Am. Soc. C.E.; 31; married; graduate of Polytechnic Institute; licensed professional engineer; 14 years varied experience sketching layouts, designing, supervising construction, estimating, contract managing, general contracting for alterations and all types high class buildings. Desires executive position or partnership with architect, engineer, or, preferably, contractor. Excellent references. Available. D-1222.

SANITARY WATER POWER ENGINEER; Jun. Am. Soc. C.E.; 27; single; 18 months sanitary inspector, assistant sanitary engineer; 6 months water power engineering, investigations, reports for power possibilities on streams, rivers; 3 months special sanitary and assistant sanitary engineer. Desires position any branch of civil engineering, preferably sanitary or water power. Available. Location immaterial. D-799.

WATER WORKS ENGINEER-CONSULTANT; M. Am. Soc. C.E.; 40; 9 years engineer-superintendent large Middle-Western water works. Designed, supervised two million dollar expansion, including large dam; 10 years U.S. Engineering Department, river and harbor work in East. Last two years consulting, promoting, designing water works, sewage disposal, paving, rates, etc. Location desired Eastern states, preferably New Jersey. D-1340.

CIVIL ENGINEER; M. Am. Soc. C.E.; 42; married; graduate; 20 years engineering experience, including municipal, railroad, water-supply, general construction, teaching, and supervision of office and field forces. Responsible connection with public utility, contractor, consulting engineer, or university desired. Location and salary open. Available immediately. D-1023.

CIVIL AND INDUSTRIAL ENGINEER; M. Am. Soc. C.E.; experience covers preliminary investigations, engineering and financial reports, design and construction of hydro-electric and steam power projects, industrial developments, and commercial buildings; also development, operation, maintenance, and consulting on chemical and industrial projects. Desires position where executive ability and contacts are required. C-6336.

JUNIOR

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 24; single; B.S. in C.E. from New York University, 1931; 1 year experience in subway construction and estimate work; desires position with construction company or in any civil engineering work. New York location preferred, but not necessary. D-1119.

GRADUATE CIVIL ENGINEER; Jun. Am. Soc. C.E.; 25; 1½ years experience with structural steel concern; 6 months railroad engineering; 9 months general surveying including highway location, real estate subdivision, and plane table surveying. Available immediately. Salary and location immaterial. D-1279.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 26; married; graduate of University of Washington, 1931; 2 years experience as rodman and recorder, U.S. Coast and Geodetic Survey; 4 months experience as assistant to chief engineer on irrigation project. Passed U.S. Civil Service examination for junior civil and junior structural engineer. Desires position in engineering work. D-1311.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; young; married; experience in railroad maintenance, bridge and tunnel work. Speaks and writes Spanish fluently. Traveling experience in South America. Will go anywhere. C-6128.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 23; single; B.S. in C.E. from Purdue University, 1931; 1 year experience in welding research and development, mathematical analysis of welded structures. Desires work in any branch of civil engineering. Available immediately. Location immaterial. References sent on request. D-1009.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 23; single; graduate; 2 years on mapping and surveying, including geodetic and cadastral work, also topographic and hydrographic surveys. Experienced chief of party. Additional experience on concrete construction, inspection, layout, estimating, and as foreman. D-1288.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 24; married; B.S. in C.E. from University of Michigan; 1 year experience in highway construction and 2 years with public utility company. Desires work in any branch of civil engineering; available immediately; location immaterial; references on request. D-1332.

SALES

SALES AND STRUCTURAL ENGINEER EXECUTIVE; M. Am. Soc. C.E.; 45; married. Background of over 20 years successful experience in engineering and sales of steel fabricating business as well as 3 years as general contractor. Desires opportunity to correspond further with an interested steel company or manufacturer of building construction material. C-5095.

TEACHING

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 30; B.S. in engineering at Princeton University, 1926; New York State license. Author of several technical papers. Unusual background of underpinning subway, tunnel, and heavy construction work. Formerly chief assistant to internationally known consulting engineer. At present holds excellent position but wishes to enter teaching. C-9232.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; M.S. degree; 2 years European study and research. Working knowledge of French and German. Last 3 years with consulting engineers, designing concrete and steel structures and supervising construction of industrial plants. Previously bridge design and construction superintendent. Experience in railroad work, water lines, sewers; 1 year teaching. Desires to teach civil engineering. B-9873.

GRADUATE CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 38; single; B.C.E., C.E., and M.S. degrees; 8 years practical experience structural design, buildings, and bridges; 8 years teaching experience general civil engineering subjects, mechanics, and mathematics. Licensed structural engineer, State of Illinois. A-1 record and references. Available at once. C-1382.

HYDRAULIC ENGINEER; Jun. Am. Soc. C.E.; 27; married; B.S. in general engineering and M.S. in hydraulic engineering from leading university. Experience in economical hydro-electric plant operation, efficiency tests, stream gaging, laboratory model studies of hydro-electric problems, other laboratory research. Desires permanent position in teaching, research, or hydro-electric field. D-1303.

GRADUATE CIVIL ENGINEER; Jun. Am. Soc. C.E.; 28; single; New York State professional engineering license; degrees of B.S., C.E., and M.A. in economics; 5 years contruction and railroad experience. Desires position teaching civil engineering subjects or economics and business courses to engineers. C-2501.

ASSOCIATE PROFESSOR STRUCTURAL ENGINEERING; Assoc. M. Am. Soc. C.E.; married; degrees B.S., M.S. in C.E., University of Illinois; 3 years experience teaching major courses, structural theory, and design in ranking university; 5 years practice on reinforced concrete and steel structures—chiefly with Waddell and Hardesty consultants—cantilever, arch suspension, movable, continuous-girder spans; technical writer; world-traveler. Desires position in leading university. D-330.

RECENT BOOKS

New books of interest to Civil Engineers, recently donated by the publishers to the Engineering Societies Library, will be found listed here. A comprehensive statement regarding the service which the Library makes available to members is to be found on page 87 of the Year Book for 1932. The statements made regarding the books are taken from the books themselves and this Society is not responsible for them.

BEMESSUNGVERFAHREN ZAHLENTAFELN UND ZAHLENBEISPIELE. By B. Löser. 4 ed. Berlin, Wilhelm Ernst & Sohn, 1932. 183 pp., diagrs., charts, tables, 10 × 7 in., paper, 6.50 rm.

A handbook of formulas and tables for the designer of reinforced-concrete structures, with numerical examples illustrating their application to a wide variety of problems. The determination of moments in framework and the design of columns and floors are discussed. This edition conforms to the rules adopted in April 1932 by the German Reinforced-Concrete Committee.

DIE FESTEN STÄDTISCHEN ABFALLSTOFFE. By Camillo Popp. München, R. Oldenbourg, 1931. 142 pp., illus., 9 × 6 in., paper, 7.60 rm.

A discussion of the advantages and disadvantages of various methods of disposing of municipal refuse and garbage. Disposal by dumping, burning, or by use as fertilizer is considered and conclusions are drawn for the guidance of municipal authorities.

ERDDRUCK, ERDWIDERSTAND UND TRAGFÄHIGKEIT DES BAUGRUNDES. By H. Krey and J. Ehrenberg. 4 ed. Berlin, Wilhelm Ernst & Sohn, 1932. 345 pp., illus., diagrs., charts, tables, 10 × 7 in., paper, 24.50 rm.

The purpose of this treatise is to assist the engineer in the practical solution of problems involving earth pressures. The principles involved are set forth, practical methods of calculation are given, and such problems as the bearing power of soils and cohesion are discussed. Extensive bibliography and collection of tables by which the usual building problems can be solved without calculation. This edition has been revised in the light of recent researches.

GRAPHS FOR ENGINEERS AND ARCHITECTS. By D. H. Lee. London, E. & F. N. Spon, Ltd.; New York, Spon & Chamberlain, 1932. 88 pp., diagrs., charts, tables, 11 × 9 in., leather, $2.

These graphs, which treat of a variety of matters of building design and construction, are intended to be used in quick designing and in checking calculations. Graphs for determining the safe loads on brick piers and steel, timber, and reinforced-concrete beams, stanchion stresses, wind stresses, panel loads, floor constructions, and similar problems are given, and their uses discussed.

INTERNATIONAL ASSOCIATION FOR BRIDGE AND STRUCTURAL ENGINEERING. Publications, vol. 1, 1932. Published by the General Secretariat, Swiss Federal Institute of Technology, Zurich. 517 pp., illus., diagrs., charts, tables, 10 × 7 in., paper, 25 Swiss fr. to members; 35 Swiss fr. to non-members.

Among the aims of this association is the publication of a periodical containing scientific articles on structural engineering. This first number contains 39 articles upon such topics as the use of welding in building, the design of slabs and girders, wind pressure, earth pressures, bridge design, etc. The papers are in French, English, or German, with English summaries.

PLANE AND GEODETIC SURVEYING. Vol. 1, Plane Surveying. By D. Clark. 2 ed. London, Constable & Co., 1932. 482 pp., diagrs., charts, tables, 9 × 6 in., cloth, 24s.

This work is intended for the engineer in the field and also the college student. The present volume covers in a thoroughly practical way the more common surveying operations of the engineer, with emphasis on the greatest accuracy attainable. Special attention is given to instrumental errors. Bibliographies on various subjects are given.

RECENT ADVANCES IN TOWN PLANNING. By T. Adams and others. New York, Macmillan Co., 1932. 400 pp., illus., diagrs., tables, 10 × 7 in., cloth, 25s.

Recent advances and tendencies of town planning in its three phases of law, practice, and design are described in this book, which aims to give a comprehensive survey of all aspects of the subject. Conditions in Great Britain are given special consideration, although conditions in America and other countries are also considered. The illustrations add to the value of the book, and there is a useful bibliography of books and reports.

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Selected items from the current Civil Engineering Group of the Engineering Index Service, 29 West 39th Street, New York, N.Y. Every article indexed is on file in The Engineering Societies Library, one of the leading technical libraries of the world. Some 2,000 technical publications from 40 countries in 20 languages are received by the Library and are read, abstracted, and indexed by trained engineers. With the information given in the items which follow, you may obtain the article from your own files, from your local library, or direct from the publisher. Photoprints will be supplied by this library at the cost of reproduction, 25 cents per page, or technical translations of the complete text may be obtained when necessary at cost.

BRIDGES

BASCULE, SAN FRANCISCO, CALIF. Third Street Bridge at San Francisco, R. P. Bryan. *West. Construction News and Highways Bldr.*, vol. 7, no. 12, June 25, 1932, pp. 357–363. New Strauss heel-trunnion single-leaf bascule bridge at Third St., San Francisco, which will provide 103-ft. clear channel between fenders; dismantling old bridge; drilling methods; arrangements for sharpening steel.

CANADA. Reinforced Concrete Bridge at Galt, Ont., A. B. Crealock. *Can. Engr.*, vol. 62, no. 23, June 7, 1932, pp. 9–11. Design and construction of highway bridge having two bowstring spans each 94 ft. 10½ in. long.

CONCRETE ARCH, CONSTRUCTION. Eliminating Initial Stresses in Concrete Arch Bridges, H. E. Steinberg. *Concrete and Constr. Eng.*, vol. 27, no. 6, June 1932, pp. 326 and 327. Review of Considère, Freyssinet, and other methods for counteracting initial stresses due to the shrinkage of concrete in hardening, and to other causes.

CONSTRUCTION, INDIA. Bridge Construction with Particular Reference to Foundations in Indian Conditions, C. I. Stabler. *Indian Eng.*, vol. 91, no. 25, June 18, 1932, pp. 495–497 (1 supp. plate). Construction of foundations; trial pits and borings; types of foundations; ordinary foundations requiring no cofferdam or shoring; open foundations requiring shoring. (To be continued.)

DESIGN. Factors Governing Design of Large Steel Bridge Projects, W. N. Elgood. *Structural Engr.*, vol. 10 (New Series), no. 7, July 1932, pp. 288–293. Financial considerations; economic design; floor system; estimating costs; foundations; weight of spans; girders and trusses; selection of type of structure.

DURABILITY. Ueber die Lebensdauer eiserner Bruecken (Stahlbruecken), F. Brunner. *Montanistische Rundschau*, vol. 24, no. 12, June 16, 1932 (supp.), pp. 2–5. Life of steel bridges; review of historical data, showing that average life of steel bridge is 50 to 100 years; comparison of maintenance costs of steel and reinforced-concrete bridges.

FOUNDATIONS, TESTING. Improved Clay Sampler Used for Bay Bridge Borings. *Eng. News-Rec.*, vol. 108, no. 25, June 23, 1932, pp. 891 and 892. Device for removing specimens of clay from foundation drill holes developed in connection with exploration program for the San Francisco-Oakland Bridge; tool permits taking samples 1 ft. long and 5 in. or 2 in. in diameter; device has been used to a depth of 273 ft. below water level; handling samples.

MASONRY ARCH, CANADA. Bridge of Reinforced Brickwork Without Falsework, W. R. McCaffrey. *Contract. Rec.*, vol. 46, no. 27, July 6, 1932, pp. 764 and 766. Design and construction of reinforced brick arch bridge at Melrose Country Club, near Philadelphia, Pa., having a single span of 42 ft. 7 in. and a rise of 6 ft.; erection methods and centering details; labor, time, and materials; future of reinforced brick masonry.

RECONSTRUCTION. Interesting Bridge Replacement Operation. *Engineer*, vol. 153, no. 3988, June 17, 1932, p. 671. Replacement of old railway bridge over Taro, on Sarzano-Parma line; old bridge consisted of seven iron spans, each 50 m. in length, between centers of piers; unusual difficulties overcome in substitution of seven iron spans; description of work.

STEEL, WELDING. Electric Arc Welding as Applied to Railway Bridges, G. R. Cockburn. *Inst. Engrs. Australia Journal*, vol. 4, no. 4, Apr. 1932, pp. 113–125. Strengthening steel, Pratt truss, railway bridge, 454 ft. long, over Hunter River at Singleton, New South Wales, by means of electric-arc welding, using bare and covered electrodes with fillet welds varying from ¼ in. to 1¼ in.; strengthening of railway bridges at Haslem's Creek, Duck River, etc.; examination of welders; deflection tests; inspection.

STRENGTHENING. C. P. R. Bridges Rebuilt for Heavier Traffic. *Contract Rec.*, vol. 46, no. 24, June 15, 1932, pp. 679–682. Strengthening and reconstruction of steel truss railroad bridge over the Grand River, on Galt Subdivision, Ontario District, also of smaller bridges of the Canadian Pacific Railway lines; falsework design.

STRUCTURAL STEEL, WELDING. Recent Welding Developments in Great Britain. *Civ. Eng. (Lond.)*, vol. 26, no. 311, May 1932, pp. 28–30. Features of several recent jobs, including 160-ft. welded girder; all-welded building; welding in reconstruction and repair of bridges on L. N. E. Railway system; strengthening of Southern Railway Co.'s Ryde Marine Pier; strengthening cast-iron bridge near Worcester.

SUSPENSION, FRANCE. Le nouveau pont métalliqué de l'ile Séguin sur la Seine, à Billancourt, près Paris. *Génie Civil*, vol. 52, no. 25, June 18, 1932, pp. 609–612, 1 supp. plate. Construction of highway bridge of modified suspension-cantilever design, having a main span 74.4 m. long, a width of about 12 m., and a total length of 143.15 m. over the Seine River at Billancourt near Paris.

VANCOUVER, B.C. Burrard Street Bridge at Vancouver, B.C., J. R. Grant. *West. Construction News and Highways Bldr.*, vol. 7, no. 12, June 25, 1932, pp. 364–366. Description of concrete and steel highway bridge 2,817 ft. long, having a main through-truss span 315 ft. long; steel erection methods on deck and through-truss spans; approach-viaduct construction.

VIADUCTS, CONCRETE. Column Forms, Sawed in Half, Are Placed by Crane. *Construction Methods*, vol. 14, no. 6, June 1932, pp. 36–39. Construction of reinforced concrete bridge, costing $2,500,000, and viaduct across the Los Angeles River at Los Angeles; total length is 3,546 ft., including two 149-ft. bowstring steel arches.

BUILDINGS

CONSTRUCTION. Replica of Federal Hall Built in 31 Days. *Construction Methods*, vol. 14, no. 6, June 1932, pp. 28–30 and 18 and 19. Historic structure reproduced in replica in Bryant Park, New York City, 59 by 95 ft. in plan and 100 ft. from ground level to top of cupola; timber frame sheathed both inside and outside with ⅜-in. fiber-incased gypsum sheets nailed to studding and finished on outside with false stone composition; several of roof trusses are 60 ft. in length.

EARTHQUAKE EFFECTS. Effects of Earthquakes on Structures—1, A. S. Mitchell. *Structural Engr.*, vol. 10 (New Series), no. 7, July 1932, pp. 294–298. Review of methods of estimating forces to be resisted in earthquakes; experience in recent New Zealand earthquake; structures with brick bearing walls; composite brick construction; reinforced-concrete construction; engineering structures.

VIBRATIONS. Multiple Periods in Vibration of San Francisco Buildings. *Eng. News-Rec.*, vol. 108, no. 25, June 23, 1932, p. 886. Observations on 15 high structures show short-period movements in addition to fundamentals; marked vertical component observed in several of them.

CONCRETE

ADMIXTURES. Do Admixtures Have Legitimate Place in Making of Concrete? L. A. Perry. *Pit and Quarry*, vol. 24, no. 7, June 29, 1932, pp. 38–40. General critical review of the problem; conservatism urged in the use of admixtures.

AGGREGATES, STONE SAND. Investigation of Gradation of Stone Sand for Concrete, A. T. Goldbeck. *Crushed Stone Journal*, vol. 8, no. 4, May–June, 1932, pp. 20–23 and 27. Testing procedure; gradations of stone sands used; design of concrete mixtures; data on concrete pavement mixes having approximately constant strength made with 17 gradations of stone sand; relation between modulus of rupture and fineness modulus; relation between cement content and fineness modulus.

BEAMS, CONCRETE. Chart Determines Shear Function in Concrete Beams. *Concrete*, vol. 40, no. 7, July 1932, pp. 20 and 21. Reproduction of Jensen chart.

COLUMNS, CONCRETE, DESIGN. Bending Stress in Concrete Columns, F. A. Hill. *Concrete*, vol. 40, no. 6, June 1932, pp. 31 and 32. Series of tables giving section coefficients for rectangular concrete beams, rectangular concrete columns, and circular concrete columns; coefficients for radius of gyration of concrete columns.

CONSTRUCTION, FORMS. Design of Concrete Form Work, J. Tucker. *Boston Soc. Civ. Engrs.—Journal*, vol. 19, no. 5, May 1932, pp. 304–310. Treasurer of Tucker Concrete Form Co. discusses the principles of the practical design of concrete form work; faults in design and construction.

CONTROL. What Are the Determining Factors Which Constitute Concrete Control? T. C. Powers. *Pit and Quarry*, vol. 24, no. 6, June 15, 1932, pp. 29–32. Specifications; twofold purpose of control; control when fixed quantity of cement is used with each batch; relation between water requirement and consistency; control of coarse and of fine aggregates; control of placing; making figures agree with facts.

CRUSHED STONE PLANTS, EQUIPMENT. Preparation and Grading of Concrete Aggregates, W. T. Miller. *Civ. Eng. (Lond.)*, vol. 26, no. 311, May 1932, pp. 33–39. British equipment and plants for the production of crushed stone; primary jaw crushers; gyratory crushers; disk crushers; gear-driven and high-speed rolls; elevators; conveyor plant; screening plants; recent crushing-plant contracts; layout of Hadfield crushing plant at Cauvery Metur Dam.

CULVERTS, CONCRETE. In Rough Country Paver Poured Long Culvert. *Contractors and Engrs. Monthly*, vol. 24, no. 6, June 1932, pp. 15-16 and 31. Construction of three box culverts, from 410 ft. to 3,900 ft. long; standardized forms reduced the cost of labor and lumber; Multi-Foote 27–3 paver was used for placing concrete on all of these culverts.

CURING. High-Pressure Steam Accelerates Rate of Hardening. *Concrete*, vol. 40, no. 7, July 1932, pp. 19 and 20. Report on tests made by Lehigh Portland Cement Co. investigators; discussion of results.

DISINTEGRATION. Concrete Test Blocks Exposed to Sea Water. *West. Construction News and Highways Bldr.*, vol. 7, no. 12, June 25, 1932, pp. 367 and 368. Results of examination of concrete specimens which have been wholly or partially submerged in ocean water off Los Angeles Harbor.

FROST EFFECT. Resistance of Concrete to Frost Action, F. H. Jackson and G. Werner. *Nat. Sand and Gravel Bul.*, vol. 13, no. 6, June 1932, pp. 9–13. Previously indexed from *American Concrete Inst.—Journal*, Apr. 1932.

HOOVER DAM PROJECT, CONSTRUCTION. Low-Level Concrete Plant for Hoover Dam, J. P. Yates. *West. Construction News and Highways Bldr.*, vol. 7, no. 2, June 10, 1932, pp. 316–323. Description of a plant for placing 4,500,000 cu. yd. of concrete; plant has four 4-yd. mixers, but can be extended to provide for a total of six 4-yd. mixers, occupies an area of 9,000 sq. ft., and extends 117 ft. in height; structure is largely built of steel; two parallel 42-in. conveyors, each with a capacity of 750 tons, are 519 ft. long and the vertical lift is 150 ft.

MIXING. Coordination of Basic Principles of Concrete Mixtures—11, J. A. Kitts. *Concrete*, vol. 40, no. 6, June 1932, pp. 14–16. Absolute volume as a basic measure of ingredients; recognition of absolute volume measure in concrete physics; work of Feret; Talbot and Richart's conclusions; Walker's yield of concrete. (Continuation of serial.)

READY-MIXED CONCRETE PLANTS, MASSACHUSETTS. Other Operators' Errors Studied Before This Concrete Plant Was Built. *Pit and Quarry*, vol. 24, no. 5, June 1, 1932, pp. 22–24 and 30. Equipment and method of operation of the

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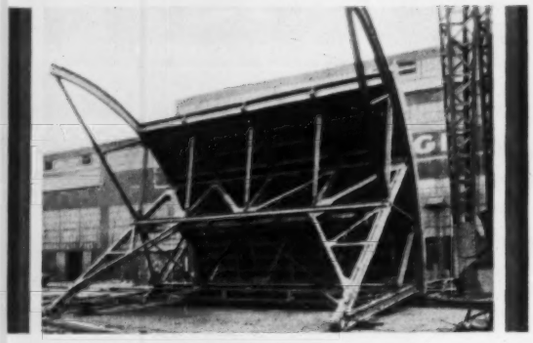

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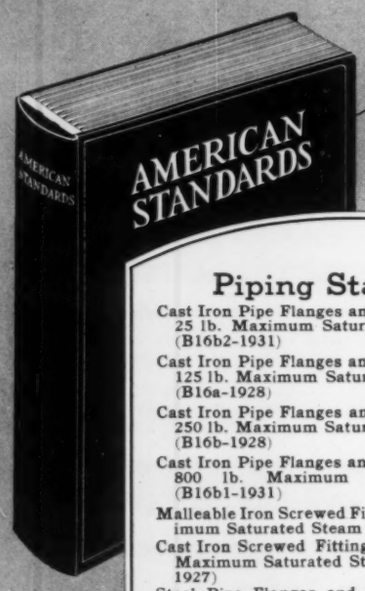

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Continued on page 16

cently completed plant of the Worcester Transit Concrete, Inc., at Worcester, Mass., with special reference to winter-plan operation; special hopper cars used to speed cement unloading.

STONE SAND. Use of Manufactured Stone Sand, R. L. Fox. *Crushed Stone Journal*, vol. 8, no. 4, May–June 1932, pp. 13–17. Experience of city engineer of Bethlehem, Pa., and observations on use of stone sand for pavement construction by highway departments of Maryland, Connecticut, Michigan, New York, Georgia, and other states; sieve analysis of stone sand; absorption tests; tests upon cores drilled from completed work.

STRENGTH. Betonguete und Festigkeitsfragen, L. David. *Tonindustrie-Ztg.*, vol. 56, no. 44, May 30, 1932, pp. 560–562. Problems of quality and strength of concrete; examples of concrete specimens from structures 50 years old.

CONSTRUCTION INDUSTRY

BUILDING CODES. Structural Provisions of Building Codes, J. A. Wise. *Minn. Federation Arch. and Eng. Soc.—Bul.*, vol. 16, no. 5, May 1932, pp. 7–9. Statement of the opinions of Minnesota engineers concerning specific provisions relating to structures in municipal building codes; specifying unit working stresses; columns; wind loads.

COSTS. Current Construction Unit Prices. *Eng. News-Rec.*, vol. 108, no. 25, June 23, 1932, p. 905. Unit prices bid on San Gabriel Dam No. 2, California, rockfill, 245 ft. high; also highway and bridge work in State of Washington.

UNITED STATES. Construction During First Half of 1932. *Eng. News-Rec.*, vol. 109, no. 2, July 14, 1932, p. 55. Statistical review; volume of private, public, and total construction for eight years, with 1932 estimated; construction contracts during 1932.

WESTERN STATES. Units Bid Summary. *West Construction News and Highways Bldr.*, vol. 7, no. 2, June 10, 1932, pp. 340, 342, 344, 346, 348, 350, and 352. Unit bids on street and road work, sewer construction, bridge and tunnel construction, etc., in California, Oregon, Utah, and other Western states.

DAMS

CONCRETE GRAVITY. Construction of Gunong Pulai Reservoir, Singapore, J. S. Jackson. *Water and Water Eng. (Special Inst. No.)*, vol. 34, no. 404, Midsummer 1932, pp. 266–274 (discussion) 274–276. Construction of water supply reservoir for Singapore on the Malay Peninsula, having a main dam of 133 ft. maximum height and 470 ft. long, built of concrete faced with granite rubble masonry; placing of concrete in dam.

CONSTRUCTION. Producing Aggregates and Concrete for Thomas W. Koon Dam. *Pit and Quarry*, vol. 24, no. 7, June 29, 1932, pp. 19–22. Description of concrete plant, automatic proportioning apparatus, and method of construction of dam described in previously indexed articles.

HOOVER DAM PROJECT, CONCRETE CONSTRUCTION. Concrete for Hoover Dam—1, W. R. Nelson. *Reclamation Era*, vol. 23, no. 5, May 1932, pp. 94–96. Concrete aggregates; Arizona gravel deposit; sand washing; screening plant details.

HYDRAULIC STRUCTURES, EUROPE. European Hydraulic Structures Design, E. C. Molke. *Can. Engr.*, vol. 62, no. 24, June 14, 1932, pp. 5–8. Practice in gate design at some low-head plants in Switzerland, Germany, and France; segment weirs; sector gates; bear-trap dams; roller dams; layout of power plants.

INDIA. Cauvery Metur Project. *Civ. Eng. (Lond.)*, vol. 26, no. 311, May 1932, pp. 40–45. Temporary through joints, or construction gaps; maximum estimated flood; irrigation areas; critical velocities for design of Grand Anicut canal system; cross drainage works; "Siphon Well" drop; change of site; project is now under execution; increased storage; canal systems; dam and its foundations; sluices.

TUNNELS. Driving 56-Ft. Tunnels for Hoover Dam. *Eng. News-Rec.*, vol. 108, no. 24, June 16, 1932, pp. 853–857. Construction of four concrete-lined diversion tunnels, 56 ft. in diameter and about 4,000 ft. long; unique drilling carriages developed for enlarging operations; rounds average 16 ft. and break 1,000 cu. yd. of rock; 438,485 cu. yd. excavated in one month; no trouble experienced in trucking operations for removal of tunnel muck; ventilation; concrete lining.

FLOOD CONTROL

RESERVOIRS. Performance of Reservoir Subjected to Flood, H. H. Jeffcott. *Royal Dublin Soc.—Sci. Proc.*, vol. 20, no. 13–20, May 1932, pp. 135–141. Outline of mathematical method of computing the capacity of spillways of flood-control reservoirs and also the flood-retarding effect of reservoirs.

FOUNDATIONS

PILES, STEEL. Extra Long Cylinder Piles Carry Column Loads to Rock, W. T. McIntosh. *Eng. News-Rec.*, vol. 108, no. 1, July 7, 1932, pp. 5–7. Description of steel cylinders, driven from 60 to 140 ft. to rock and blown out by compressed air, which form foundations of the Starrett-Lehigh Building in New York; tests show settlement of $\frac{1}{8}$ in. under load designed one and a half times.

TESTING. Driven Casing Extracts Long Earth Cores. *Construction Methods*, vol. 14, no. 6, June 1932, p. 25. Description of a patented method for obtaining vertical sections of subsoil, 30 to 40 ft. long, which consists of driving successively 4 by 4-in. angle and 7-in. channel, and then withdrawing them simultaneously to enclose triangular core of earth; special equipment developed at West Side sewage treatment plant of Sanitary District of Chicago.

HYDRAULIC ENGINEERING

MEASUREMENT. Ancient and Modern Methods of Water Measurement, F. C. Lea. *Water and Water Eng. (Special Inst. No.)*, vol. 134, no. 404, Midsummer 1932, pp. 276–278 and 280–290. Review of formulas and instruments for measuring flow of liquids; use of models to determine coefficients of discharge of weirs; siphon spillways; Venturi meter; orifice diaphragm; pitot tube; chemical and electrical methods of measuring flow of water; salt velocity method; Venturi flume; examples of dimensional theory.

SPILLWAYS, MODEL TESTING. Model Tests Verify Design of Madden Dam Spillway. *Eng. News.-Rec.*, vol. 109, no. 2, July 14, 1932, pp. 42–44. Extensive studies made in Colorado laboratory to check design of sloped apron at the toe of gravity-section ogee spillway of 400 ft. net length, designed to take care of a discharge of 280,000 sec-ft. on the Chagres River in the Panama Canal Zone.

HYDRO-ELECTRIC POWER PLANTS

CONSTRUCTION. Special Features on Large Construction Job, J. B. Macphail. *Contract. Rec.*, vol. 46, no. 23, June 8, 1932, pp. 653–655. Description of road and bridge work preliminary to the construction of the Shawinigan Water and Power Company's new hydro-electric plant on the St. Maurice River at Rapide Blanc, 30 miles above La Tuque, Que.

RUSSIA. American Methods Win Fight to Control Russian River. *Eng. News-Rec.*, vol. 108, no. 25, June 23, 1932, pp. 877–883. History and method of construction of the largest hydro-electric power plant in the world at Kichkas on the Dnieper River, in Ukraine, including concrete spillway dam, about 140 ft. high, surmounted with gates, each with span of 13 m.; construction operations; deep excavation; expansion joints; foundation grouting; labor problems.

HYDROLOGY, METEOROLOGY, AND SEISMOGRAPHY

DROUGHT. Drought of 1930 and Surface Water Supplies in Illinois, W. D. Gerber. *Am. Water Works Ass'n Journal*, vol. 24, no. 6, June 1932, pp. 840–853. Review of effect of the 1930 drought on Lake Michigan, Mississippi River, Ohio and Wabash rivers, and interior Illinois streams.

FLOOD DISCHARGE, HAWAIIAN ISLANDS. Hawaiian Storm Produces Flood of 5,400 Sec-Ft. per Square Mile. *Eng. News-Rec.*, vol. 108, no. 25, June 23, 1932, p. 883. Data on remarkable flood from 2.35 sq. miles of watershed area above recently completed Alexander Dam, on island of Kauai.

METEORITES, NEBRASKA. Metallic Meteorite from Ogallala, Nebraska, H. H. Nininger. *Am. Mineralogist*, vol. 17, no. 6, June 1932, pp. 221–225. Notes on physical and chemical examination of specimen weighing 3,300 grams.

RIVERS, AFRICA. Nile Basin, H. E. Hurst and P. Phillips. *Ministry Pub. Works, Egypt—Physical Dept. Paper No. 26*, vol. 1 (Gov. Press, Cairo, 1931), 144 pp., 117 supp. plates. P. T. 50. History of discovery in Nile Basin in modern times; description of Nile Basin; hydrology; irrigation; meteorology; rainfall; evaporation; changes of climate; topography of Lake Plateau Basin; Bahr el Jebel, Bahr el Zeraf and Sudd Region; Bahr el Ghazal; Sobat Basin; White Nile. Bibliography.

RAIN AND RAINFALL, MEASUREMENT. Rainfall—V. *Water and Water Eng.*, vol. 34, no. 405, June 20, 1932, pp. 298–301. Variations in rainfall over the British Isles; residual-mass percentage curve based on 44-year normal, from 1887 to 1930.

LAND RECLAMATION AND DRAINAGE

CULVERTS, CONSTRUCTION. Concrete Culverts Lengthened with Corrugated-Iron Pipe, H. A. Pabst. *Eng. News-Rec.*, vol 108, no. 24, June 16, 1932, p. 859. Widening a section of highway between Benson and Hancock, Minn., requiring a lengthening of 23 concrete culverts at both ends, a total of 18 ft.; connecting metal pipe with concrete-box culvert end by 24-in. pipe with expansible end.

MATERIALS TESTING

CEMENT. Practical Methods for Testing Masonry Mortar Cements, F. O. Anderegg. *Rock Products*, vol. 35, no. 11, June 4, 1932, pp. 28-30. Comparative tests for masonry cements; workability; adhesion to masonry units; watertightness; weather resistance; flexibility; shrinkage; compressive strength; efflorescence; fading; staining; mortar color pigments.

CONCRETE. Making, Curing, and Testing of Concrete Cylinders, D. B. Rush. *Concrete*, vol. 40, no. 7, July 1932, pp. 15 and 16. Accuracy of cylinder tests; types of cylinder molds; metal molds recommended; making and storing cylinders; curing; information from job-stored cylinders; inspection engineer should make cylinders.

CONCRETE AGGREGATES, STONE SAND. Dolomite Sand in Concrete for Henley Street Bridge, Knoxville, Tennessee, L. M. Dow. *Crushed Stone Journal*, vol. 8, no. 4, May–June 1932, pp. 10–12 and 17–19. Results of laboratory tests of stone sand for use in construction of reinforced concrete arch bridge costing $1,000,000.

METAL, CREEP. Creep of Metals, A. Nadai. *Am. Soc. Mech. Engrs.—Advance Paper*, meeting June 23–25, 1932, 8 pp., 1 supp. plate. Idealized cases of plastic flow reviewed with primary intention of throwing some light on the effect of speed of yielding on other quantities to be observed when metallic bars are permanently stretched, twisted, or bent at normal, or at elevated, temperatures.

ROAD MATERIALS, TESTING. Adaptation of Stability Test to Include Coarse Aggregate Asphalt Paving Mixtures, P. Hubbard and F. C. Field. *Eng. and Contracting*, vol. 71, no. 6, June 1932, pp. 160 and 161. Asphalt institute mold and testing ring for coarser mixtures; hot-mix asphalt paving mixtures; cold-mix paving mixtures. Before Asphalt Paving Technologists Ass'n.

GIBRALTAR. Decade of Municipal Work and Improvement in Gibraltar, W. H. Pearce. *Surveyor*, vol. 81, no. 2109, June 24, 1932, pp. 625–627. Review of routine engineering work; construction and maintenance of concrete-lined mountain slopes acting as artificial watersheds for city's water works.

PORTS AND MARITIME STRUCTURES

DESIGN. Sul calcolo delle sollecitazioni nelle opere marittime di difesa a parete verticale, S. V. Fago. *Ingegnere*, vol. 6, no. 4, Apr. 1932, pp. 248–259. Theoretical analysis of stresses in vertical face breakwaters; critical review of theories advanced by Cornaglia, Benezit, and Sainflow.

GLASGOW, SCOTLAND. Recent Improvements and Reconstruction Works in Glasgow Harbor, A. C. Gardner. *Inst. Engrs. and Shipbldrs. in Scotland—Trans.*, vol. 75, part 6, Apr. 1932, pp. 459–483 (discussion) 483–484, and 486, 6 supp. plates. Recent advancements made either in way of improving facilities or developing work in connection with the river, which have now been carried on almost continuously since Clyde became a navigable highway to the sea.

LONG BEACH, CALIF. Creosoted Timber Mat Seals Breakwater in Long Beach Harbor. *World Ports*, vol. 20, no. 8, June 1932, pp. 569–573. Successful use of creosoted timber mat to seal rubble breakwater behind which the area was to be filled by fine sand and silt material dredged from deep water. From *Wood Preserving News*.

NEW YORK. Interesting Features in Construction of Inland Terminal No. 1, J. C. Evans. *World Ports*, vol. 20, no. 7, May 1932, pp. 477–489 (discussion) 489–494. Study of freight traffic in lower Manhattan up to 59th Street; general plan and equipment of New York Port Authority terminal, known as Inland Terminal No. 1, occupying a block bounded by 8th and 9th avenues and 15th and 16th streets.

PALESTINE. New Harbor at Haifa. *Engineer*, vol. 154, no. 3990, July 1, 1932, p. 16. Progress made with construction of harbor which will be the only one on the coast of Palestine; it is being built on natural blocks of quarried stone; material for reclamation will come from dredging within breakwaters.

UNITED STATES. Port of Duluth-Superior, Minn. and Wis. *U.S. Army and U.S. Shipg. Bd.—Lake Series*, no. 6, 1932, 161 pp. 14 supp. plates, 50 cents. Data showing the movement of commerce through port of Duluth-Superior; facilities available for handling traffic, and rates and charges applying against it.

RAILROADS, STATIONS, AND TERMINALS

MILAN, ITALY. La nouvelle gare de Milan, L. Gerard. *Technique des Travaux*, vol. 8, no. 5, May 1932, pp. 283–294. Description of new railroad station of Milan, Italy, similar in content to several previously indexed articles.

TRAIN SHEDS, ROOFS. Long-Span Platform Roofs for Cincinnati Station, O. S. Payzant. *Eng. News-Rec.*, vol. 108, no. 26, June 30, 1932, p. 912. Steel framing for platform roofs of Cincinnati union station is made in three-span lengths; girders are continuous over three 80-ft. spans in steel-frame flat shelter roofs; span adapted to average length of passenger cars.

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ROADS AND STREETS

AUSTRALIA. Establishment of Direct Road Communication Between Sydney and Newcastle, T. H. Upton. *Inst. Engrs. Australia—Journal*, vol. 4, no. 5, May 1932, pp. 159–174. History of road connection between Sydney and Hunter River Valley from earliest days of colony; steps taken by Main Roads Board of New South Wales, leading to development of a direct route between Sydney and Newcastle, adjacent to coast; details of concrete pavement; output of concrete mixer. (To be continued.)

CANADA. Status of Trans-Canada Highway. *Can. Engr.*, vol. 62, no. 25, June 21, 1932, pp. 15–20. Construction progress report and description of several particular sections of the system.

CONCRETE. State Highway Design Standards for Paving One-Traffic Lane, E. M. Fleming. *Concrete*, vol. 40, no. 7, July 1932, pp. 5–8. Present practice in Illinois, Maryland, Delaware, and Missouri; recommendations of Illinois highway department for single-track paving and widening.

CONSTRUCTION. Settling Fills with Explosives, L. F. Livingston. *Eng. and Contracting*, vol. 71, no. 6, June 1932, pp. 149 and 150. Blasting of highway fills; trench shooting method; drilling holes for loading; underfill method; relief method.

EMPLOYMENT UNITS. Computing Employment Units in Road Construction, G. C. Dillman. *Eng. News-Rec.*, vol. 108, no. 26, June 30, 1932, p. 922. Michigan costs and employment units for three principal classes of road work; 100 miles can be graded, or 77 miles of gravel road or 36 miles of concrete pavement can be constructed for $1,000,000.

EXPLOSIVES. How 240 Tons of Explosives Were Used Excavating California Highway, C. S. Pope. *Roads and Streets*, vol. 75, no. 7, July 1932, pp. 299–301. Construction of re-located U.S. Highway, Route 99, traversing rocky sides of Shasta Canyon in northern California; various types of holes prepared by explosives; bootleg holes; explosives used; types of powder pockets; drill bit types.

GRADING. Efficiency Studies of Highway Grading Operations, T. C. Thee. *Roads and Streets*, vol. 75, no. 7, July 1932, pp. 289–294. Results of studies of grading operations in 1931 on Federal Aid projects 433-AB and 385-BC, in Washburn and Barron counties, Wisconsin.

HIGHWAY ENGINEERING. Recent Developments in Design and Construction of Highways, R. W. Coburn. *Boston Soc. Civ. Engrs.—Journal*, vol. 19, no. 5, May 1932, pp. 281–290 (discussion) 290–292. Boston and Worcester Turnpike; elimination of railroad and highway grade crossings; traffic circles; by-pass routes; preliminary details of design; non-skid surfaces; low cost secondary roads; concrete road construction; treatment of swamps; modern machinery and bid prices.

HIGHWAY SYSTEMS, UNITED STATES. Building Mountain Highways in Big Smokies. *Eng. News-Rec.*, vol. 109, no. 2, July 14, 1932, p. 45. Features of skirting highways and across-park road built by adjoining states is first step in system of roads and trails for Great Smoky Mountain National Park in eastern Tennessee and western North Carolina.

MACHINERY. Spreading and Finishing Equipment. *Can. Engr.*, vol. 62, no. 25, June 21, 1932, pp. 21 and 22. Development of mechanical finishing machines and tampers for concrete and bituminous pavements on basis of a survey made by a joint committee of the American Association of State Highway Officials and the American Road Builder's Association.

MAINTENANCE AND REPAIR. Recent Developments in Maintenance and Use of Highways, G. H. Delano. *Boston Soc. Civ. Engrs.—Journal*, vol. 19, no. 5, May 1932, pp. 293–301 (discussion) 301–303. Treatment of road surfaces; mud pumps; permanent patching; snow removal; traffic; white lines; road-sides; selective trimming.

MATERIALS, ASPHALT. Asphalt and Its Use for Roads, C. M. Baskin. *Contract. Rec.*, vol. 46, no. 26, June 29, 1932, pp. 753–756. Study of the chemistry of asphalt; significance of source and effect of modern processes on present-day specifications. Before Ass'n of Asphalt Technologists. (To be continued.)

OHIO. Ohio Builds Road-Mix Job with New Tools, H. S. Perry. *Roads and Streets*, vol. 75, no. 7, July 1932, pp. 295 and 296. Description of hone and mixer developed for low-cost road work.

SCOTLAND. County Road Practice in Scotland, G. S. Barry. *Surveyor*, vol. 81, no. 2108, June 1, 1932, pp. 605–607. Extracts from paper presented at the annual meeting of the Institution of Municipal and County Engineers; contract system; departmental labor system; quarries and production of road material; surface treatment; coatings; grouting; use of asphalt; footpaths; foundation work.

SURVEYS. Uses of Traffic Census, E. W. James. *Highway Engr. and Contractor*, vol. 40, no. 9, June 1932, pp. 9, 10, 41 and 43. Information furnished by traffic census to aid engineer in deciding on details of construction, in arranging building programs, determining priority of construction, and to assist administrative official in deciding on policies concerning highway financing, extension of mileage, distribution of taxation, and related matters; practice of states of Pennsylvania and Michigan.

TESTING. National Crushed Stone Association Circular Testing Track, A. T. Goldbeck, J. E. Gray and L. L. Ludlow, Jr. *Crushed Stone Journal*, vol. 8, no. 4, May–June 1932, pp. 5–8. Description of pavement-testing apparatus consisting of a circular track, 14 ft. in mean diameter, upon which road and traffic conditions may be reproduced; use of apparatus for purpose of pavement design.

VIRGINIA. Skyline Highway Tops Blue Ridge Mountains. *Eng. News-Rec.*, vol. 109, no. 1, July 7, 1932, pp. 10–12. Location and construction of skyline drive on top of Blue Ridge Range in Shenandoah National Park, Va., commanding panorama of Shenandoah and Piedmont valleys for 100 miles, grading sections to Federal standards for national parks roads; rough grade to profile with tractor-blade grader; steep side-hill fills are held within bounds by batter toe walls of hand-laid rubble.

SEWERAGE AND SEWAGE DISPOSAL

FAILURE. Pipe Sewer Failures in Chicago Region, P. E. Green. *Eng. News-Rec.*, vol. 109, no. 1, July 7, 1932, pp. 18 and 19. Study of the causes of the failure of sewer pipe lines in Homewood, Ill., and in Hazelhurst, Ill.; pipe sewer must be protected from too wide a trench or yielding bottom; concrete cradling is necessary; pre-molded bituminous joints must be set.

PLANTS, GLASGOW, SCOTLAND. Sewage Purification in Glasgow, W. S. Tennant. *Surveyor*, vol. 81, no. 2108, June 17, 1932, pp. 609 and 610. Extracts from paper presented at Annual Meeting of Institution of Municipal and County Engineers; features of Dalmarnock works, Dalmuir works, and Shieldhall works; sludge disposal.

SLUDGE. Removal of Sand-Bed Sludge and Its Sale for Fertilizer. *Eng. News-Rec.*, vol. 109, no. 2, July 14, 1932, pp. 41 and 42. Sewage sludge from Imhoff tanks at Dayton, Ohio, is dried in eleven glass-covered beds with a total area of 105,600 sq. ft.; 0.4 sq. ft. per capita; summary of operating record, 1931.

UNITED STATES. American Sewage Treatment as Seen by English Expert, J. D. Watson. *Eng. News-Rec.*, vol. 109, no. 2, July 14, 1932, pp. 54 and 55. Impressions following an extended trip through Eastern and Middle Western plants.

STRUCTURAL ENGINEERING

BEAMS, DESIGN. Analysis of Statically Indeterminate Beams by Theorems Two and Three Moments, A. L. Miller. *Univ. Wash.—Eng. Experiment Station Series—Bul., No. 64*, May 1, 1932, 51 pp. Theoretical mathematical discussion of flexural elasticity and analysis of statically indeterminate beams; derivation and applications of theorem of two moments; supported cantilever beam; fixed-end beam with settlement; theorems of three moments; two-span and four-span beams; beams of variable cross section; comparison of various methods.

CONCRETE DESIGN. Conference on Bridge and Structural Engineering. *Concrete and Constr. Eng.*, vol. 27, no. 6, June 1932, pp. 328–338. Abstracts of papers presented before the first congress of the International Association for Bridge and Structural Engineering: Large-Span Girder Bridges in Germany, H. Spangenberg; Steel Encased in Concrete, L. Santarella; Mesh-Reinforced Slabs, W. Gehler.

TRAFFIC CONTROL

SOUTH AFRICA. Traffic Control in City of Johannesburg, F. W. Norman. *Surveyor*, vol. 82, no. 2110, July 1, 1932, p. 4. Parking accommodation; automatic traffic signals; traffic signs.

WATER PIPE LINES

CONCRETE-STEEL. Reinforced Concrete-Steel Water Mains, C. J. Desbaillets. *Water and Water Eng.*, vol. 34, no. 405, June 20, 1932, pp. 315 and 316. Installation of 34-in. Bonna pipe for high-level distribution system at Montreal; characteristics of design.

INTAKES. Huge Derrick Boat Built to Handle Toronto Intake Pipes. *Contract. Rec.*, vol. 46, no. 25, June 22, 1932, pp. 708 and 709. Description of electrically operated derrick boat, "Foundation Scarboro," which will take care of 100-ft. lengths of steel and concrete pipes weighing 250 tons per section; pumping equipment.

WATER RESOURCES

FLORIDA. Everglades Domestic Water Supply, C. F. Warner. *Am. Water Works Ass'n Journal*—vol. 24, no. 6, June 1932, pp. 885–890. Physical and chemical properties of underground and surface drainage waters of Florida Everglades territory outside of the economic reach of Lake Okeechobee; chemical analyses and methods of treatment of water derived from Everglades drainage canals.

MONTREAL. Montreal Water Supply, C. J. Desbaillets. *Am. Journal Public Health*, vol. 22, no. 6, June 1932, pp. 627–632. Brief historical review of Montreal water supply developments; characteristics of pumping stations; high pressure pumping zone; water chlorination.

WATER TREATMENT

ANALYSIS. Quality of Water of Colorado River in 1928–1930, C. S. Howard. *U.S. Geol. Survey Water-Supply Paper 638—D* 1932, pp. 145–168. Results obtained in continuation of study of Colorado River begun in 1925; analyses of composites of daily samples collected at gaging stations on Colorado River, Green River, and on San Juan River; utilization of water of Colorado River and its tributaries; suspended matter.

FILTRATION, AERATION. Filtration Plants J. R. Baylis. *Water Works and Sewerage*, vol. 79, no. 6, June 1932, pp. 105–108. Effect of films at water-gas interface; thickness of films; gas molecules passing through water-air films; escape of dissolved gases through surface films; gases which require high percentage saturation to produce taste or odor may be easily removed by aeration; experiments on reduction of CO_2; large water-air surface essential for efficient aeration. Bibliography.

GREAT BRITAIN. Chemical and Bacteriological Examination of London Waters, A. Houston. *Water and Water Eng.*, vol. 134, no. 405, June 20, 1932, pp. 292–294. Abstract and critical review of the 26th Annual Metropolitan Water Board Report previously indexed.

OHIO. Water Softening Plant, J. A. Wade. *Water Works Eng.*, vol. 85, no. 12, June 15, 1932, pp. 738–740. Description of Ohio Water Service Company's plant at Massillon, Ohio, having a nominal capacity of 2.4 m. g. d., for purification of water having traces of hydrogen sulfide and iron; clarifying and recarbonating chambers; carbon dioxide scrubber and compressors.

PURIFICATION. Elements of Water Purification, C. K. Mathews. *Water Works Eng.*, vol. 85, no. 13, June 29, 1932, pp. 793–794. Elements of interest to man about purification plant; promotion of chemical reactions; sand filters; removal of sludge; sterilization of water; relation of pH to corrosion. Before Water Works Operators' School.

SOFTENING, SLUDGE DISPOSAL. Advantages of Mechanical Removal of Sludge in Water-Softening Plants, F. Bachmann. *Water and Water Eng.*, vol. 34, no. 405, June 20, 1932, pp. 318 and 319. Advantages of mechanically cleaned tanks; reduction of cost of bains; rate of precipitation accelerated; reduction of stream pollution, odor, and of water loss; prevention of organic decomposition; regulation of sludge removal.

UNITED STATES. Water Purification Outside of California, R. F. Goudey. *Am. Water Works Ass'n—Journal*, vol. 24, no. 6, June 1932, pp. 822—829. Results of study of seven filtration plants, ten water-softening plants, and four water systems, presenting such special problems as manganese, corrosion, and taste troubles.

WATER WORKS ENGINEERING

BOSTON, MASS. Boston Pushes Its Water Program, M. R. Budd. *Explosives Engr.*, vol. 10, no. 7, July 1932, pp. 208–213. Outline of water development program of Boston's Metropolitan District Water Supply Commission; construction features of Wachusett-Coldbrook Tunnel, Coldbrook-Swift Tunnel, Swift River Reservoir, and Swift River Diversion Tunnel, with special reference to use of explosives in tunneling.

MANAGEMENT. Distribution System Facts and Fancies, C. Goldsmith. *Am. Water Works Ass'n—Journal*, vol. 24, no. 6, June 1932, pp. 854–858. Review of management practice; regular inspections of valves and hydrants; stocking repair parts; distribution system design; water ram.

TANKS. Walk of Reinforced Brickwork Encloses Steel Water Tank. *Eng. News-Rec.*, vol. 108, no. 25, June 23, 1932, pp. 892 and 893. Reinforced brickwork used in masonry enclosure for steel water tank, in Baltimore, Md.; capacity 4 m. g. d.; circular brick wall, 119 ft. in outside diameter, completely encloses tank, with clear space of 2½ ft. left from steel shell and wall; wall varies in thickness from 17 to 13 in.; cost $100,000.

TOWERS, CONCRETE. Steel and Concrete Water Tower, F. Hibbert. *Civ. Eng. (Lond.)*, vol. 26, no. 311, May 1932, pp. 47–49. Construction of reinforced concrete water tower 60 ft. high carrying a tank 12 ft. deep, and with a capacity of 80,000 gal.; major part of steel employed takes form of rigid pre-erected skeleton framework designed to carry temporary centering, wet concrete, and working loads incidental to construction.

UNITED STATES. A. W. W. A. Convention Proceedings. *Water Works Eng.*, vol. 85, no. 12, June 15, 1932, pp. 751–753. Discussion of numerical rating of water supplies, removal of manganese, water softening, high rate of filter wash, aeration and its uses, floc formation, and many other kindred topics. (To be continued.)

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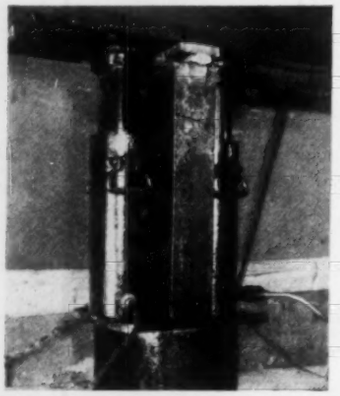

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